(12)

SSI 8144-7

September 1982

RESEARCH ON UPGRADING STRUCTURES FOR HOST AND RISK AREA SHELTERS

FINAL REPORT

for

FEDERAL EMERGENCY MANAGEMENT AGENCY
WASHINGTON, D.C. 20472

Contract No. EMW-C-0686
Work Unit 1128A

SCIENTIFIC SERVICE, INC.

UNCLASSIFIED

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE		READ INSTRUCTIONS BEFORE COMPLETING FORM
1. REPORT NUMBER SSI 8144-7	2. GOVT ACCESSION NO. AD-A121769	3. RECIPIENT'S CATALOG NUMBER
4. TITLE (and Subtitle) RESEARCH ON UPGRADING STRUCTURES FOR HOST AND RISK AREA SHELTERS		5. TYPE OF REPORT & PERIOD COVERED Final Report 6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s) R.S. Tansley, G.J. Cuzner, and C. Wilton		8. CONTRACT OR GRANT NUMBER(s) EMW-C-0686
9. PERFORMING ORGANIZATION NAME AND ADDRESS Scientific Service, Inc. 517 East Bayshore, Redwood City, CA 94063		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS Work Unit 1128A
11. CONTROLLING OFFICE NAME AND ADDRESS Federal Emergency Management Agency Washington, D.C. 20472		12. REPORT DATE September 1982 13. NUMBER OF PAGES 227
14. MONITORING AGENCY NAME & ADDRESS(if different from Controlling Office)		15. SECURITY CLASS. (of this report) 15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

Approved for Public Release; Distribution Unlimited

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

18. SUPPLEMENTARY NOTES

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

Civil Defense; crisis relocation; host area shelters; risk area shelters; upgrading

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This report present a summary of the work conducted during the first year of a five-year program. This research effort provides the engineering basis and guidance for the development of upgrading for host and risk area shelters.

This investigation is in support of current Civil Defense planning based on a policy of crisis relocation, and includes investigative efforts related to glulam timber beams, concrete connections, punching strength of reinforced

cont

DD FORM 1 JAN 73 1473 EDITION OF 1 NOV 65 IS OBSOLETE

UNCLASSIFIED

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

Block 20 (contd):

concrete slabs, and static/dynamic testing of prestressed concrete slabs.

The results of this study are being used in the development of a prediction methodology for comparative selection of shelter spaces.

(DETACHABLE SUMMARY)

SSI 8144-7 Final Report
September 1982

RESEARCH ON UPGRADING STRUCTURES FOR HOST AND RISK AREA SHELTERS

by

R.S. Tansley, G.J. Cuzner, and C. Wilton

for

Federal Emergency Management Agency
Washington, D.C. 20472

Contract No. EMW-C-0686, Work Unit 1128A

FEMA REVIEW NOTICE:

This report has been reviewed in the Federal Emergency Management Agency and approved for publication. Approval does not signify that the contents necessarily reflect the views and policies of the Federal Emergency Management Agency.

Scientific Service, Inc.
517 East Bayshore, Redwood City, CA 94063

(Detachable Summary)

RESEARCH ON UPGRADING STRUCTURES FOR HOST AND RISK AREA SHELTERS

Scientific Service, Inc. is conducting a five-year program of research on the upgrading of structures for host and risk area shelters. The purpose of the program is to provide the engineering basis and guidance for this upgrading, as well as for expedient shelter protection. This report presents a summary of the work conducted during the first year of this five-year program.

The current investigation is built on a considerable amount of previous research and testing conducted by SSI. Included in these previous investigations were the development of failure prediction methodologies for common construction types, in both "as-built" and upgraded configurations. These analysis and prediction techniques have been applied to floors and roofs constructed of many different building systems and materials, and have been verified by full- and small-scale static and dynamic testing.

This year's research effort continued with the full-scale testing of systems not previously investigated and further expanded the research into areas other than floors and roofs (such as structural connections and the punching strength of concrete slabs) that directly affect the performance of potential shelters. In order to complement one of the principal goals of the five-year program, i.e., to develop the complete methodology for the comparative selection of the best shelter spaces available, investigation of these structually related areas is required and will continue to be emphasized throughout the program.

Since 1978, a preliminary survival matrix for floors has been published, and in 1981 a similar matrix for roofs was added. As the results of the analytical and experimental programs have become available, these matrices have been continually

modified and updated. Both are again presented in this report as Tables 6-1 and 6-2, and have been revised to include the data developed and reported herein, the data from the MILL RACE event conducted in September 1981, and from recent tests conducted at the Waterways Experiment Station.

These matrices indicate the overpressure in psi at which 95% of the floor or roof systems are predicted to survive "as-built" and with various types of shoring. Those values that have been obtained from testing are indicated. The survival pressure predictions indicated for the various types of construction were determined by assuming the dead loads (load of structure itself), and increasing the design live loads by the safety factors required for design for the particular construction considered. The "as-built" survival overpressure considers the floor or roof "as-is" with no upgrading or shoring, and **ALL** assume radiation protection equal to a P_f of 100 (18 inches of earth equivalent) superimposed on the system.

Table 6-2, the survival matrix for roofs, has been revised from its last presentation. The portion of the matrix that included roofs constructed of various types of concrete construction has been deleted, and a note added to direct the user to the floor matrix for this information. This revision reflects the fact that concrete systems are rarely used for light roof loading, and when used, the roof probably has been designed as a future floor, to carry mechanical loads, or in an area with severe snow loading requirements -- making it more appropriate to use the floor matrix.

Another revision to the roof survival matrix consisted of adding an additional column headed "Load Rating". The purpose of this revision was to incorporate into the matrix the increased values of survivalbility that might be expected for roofs in locations where snow loads are required by code to be recognized in the design process.

TABLE 6-1: PRELIMINARY SURVIVAL MATRIX FOR FLOORS

Overpressures at which 95% of Floors Will Survive "As Built" and with Various Types of Shoring (All Values in psi)

3

Type of Floor Construction and Dead Load	As Built		Shoring Required											
			Midspan		1/3 Span		1/4 Span		King-Post Truss		Flange		Boxed Beam	
	Pred	Test	Pred	Test	Pred	Test	Pred	Test	Pred	Test	Pred	Test	Pred	Test
WOOD														
Light - Joist	†	*0.4*	3.3	*6.8*	8.6	*9.2*	-		1.6	*1.8*	1.1	*1.1*	1.1	*2.1*
Light - Glulam	†	*0.2*	3.3	*3.0*	8.6									
Medium - Joist	0.9	*1.5*	6.7	*8.1*	16.4	*12.9*	-		3.8		2.8		2.8	
Medium - Glulam	0.9	*1.3*	6.7	*7.0*	16.4		-							
Heavy - Plank	2.6		13.7		32.1				8.2		-		-	
STEEL, LIGHT														
Light - Open-Web Joist	0.2	*0.3*	1.0	*1.2*	2.8	*3.5*	-		1.0		-		-	
Medium - Open-Web Joist	1.4	*1.6*	3.0	*3.4*	6.6	*8.0*	-		2.5		-		-	
STEEL, HEAVY														
Light - Beam and Slab	0.1		3.1		7.9		-		-		-		-	
Medium - Beam and Slab	0.8		5.5		13.3		-		-		-		-	
Heavy - Beam and Slab	2.0		10.3		24.0		-							

Note: Overpressure values assume radiation protection equal to a P_f of 100 (18 in. of earth or equivalent) superimposed on floor. Assumed density of earth = 100 pcf.

†- Required radiation protection (P_f = 100) would cause floor to collapse.

TABLE 6-1: PRELIMINARY SURVIVAL MATRIX FOR FLOORS (contd)

Overpressure at which 95% of Floors Will Survive "As Built" and with Various Types of Shoring (All Values in psi)

Type of Floor Construction and Dead Load	As Built		Shoring Required: Midspan		Shoring Required: 1/3 Span		Shoring Required: 1/4 Span	
	Pred	Test	Pred	Test	Pred	Test	Pred	Test
CONCRETE								
Light - Single and Double Tees, One-Way Joists	0.6		4.7		11.7		-	
Light - Hollow-Core Slabs	0.6	*0.6*	4.8	*6.2*	11.7	*16.1*	-	
Light - One-Way Solid Slabs	0.9		5.0		12.0		-	
Light - Flat Slab, Flat Plate - Two-Way	0.9		5.0		12.0		21.7	
Light - Waffle Slab	0.6		4.7		11.7		21.4	
Medium - Single and Double Tees, One-Way Joists	1.3		7.6		18.0		-	
Medium - Hollow-Core Slabs	1.5	*2.2*	7.8	*10.2*	18.2	*19.8*	-	
Medium - One-Way Solid Slabs	1.7		8.0		18.4		-	
Medium - Flat Slab, Flat Plate - Two-Way	1.7		8.0		18.4		33.0	*36.5* (1) *37.0* (2)
Medium - Waffle Slab	1.3		7.6		18.0		32.6	*43.0* (1)

Note: Overpressure values assume radiation protection equal to a P_f of 100 (18 in. of earth or equivalent) superimposed on floor. Assumed density of earth = 100 pcf.

(1) Waterways Experiment Station Test

(2) MILL RACE Test

TABLE 6-1: PRELIMINARY SURVIVAL MATRIX FOR FLOORS (contd)

Overpressure at which 95% of Floors Will Survive "As Built" and with Various Types of Shoring (All Values in psi)

Type of Floor Construction and Dead Load	As Built		Shoring Required					
			Midspan		1/3 Span		1/4 Span	
	Pred	Test	Pred	Test	Pred	Test	Pred	Test
CONCRETE								
Heavy - Single and Double Tees, One-Way Joists	2.8		13.2		30.6		-	
Heavy - Hollow-Core Slabs	3.0	*5.3*	13.4	*19.9*	30.8	*30.1*	-	
Heavy - One-Way Solid Slabs	3.3	*5.9*	13.7	*17.7*	31.1	*36.2*	-	
Heavy - Flat Slab, Flat Plate - *Two-Way*	3.3		13.7		31.1		55.4	
Heavy - Waffle Slab	2.8		13.2		30.6		54.9	

Note: Overpressure values assume radiation protection equal to a P_f of 100 (18 in. of earth or equivalent) superimposed on floor. Assumed density of earth = 100 pcf.

TABLE 6-2: PRELIMINARY SURVIVAL MATRIX FOR ROOFS

Overpressure at which 95% of Roofs Will Survive
"As Built" and with Various Types of Shoring. (All Values in psi)

Type of Roof Construction	Load Rating (1)	Shoring Required: As Built Pred	As Built Test	Midspan Pred	Midspan Test	1/3 Span Pred	1/3 Span Test	1/4 Span Pred	1/4 Span Test
WOOD									
Joist	A	†	-	0.7	-	-	-	-	-
	B	†	-	1.4	*2.0* (2)	-	-	-	-
	C	†	-	2.8	-	-	-	-	-
Glulam	A	†	†	0.7	*0.5*	-	-	-	-
	B	†	†	1.4	*1.2*	-	-	-	-
	C	†	*0.3*	2.8	*2.8*	-	-	-	-
Gabled Truss	A	†	†	†	†	0.2	*0.2* (3)	0.7	*1.1*
	B	†	†	0.2	*0.5*	0.8	*1.1*	1.4	-
	C	†	*0.3*	0.8	*1.8*	1.8	2.9	2.4	-
LIGHT STEEL									
Open-Web Joist with Plywood Deck	A	†	-	†	-	0.2	-	-	-
	B	†	-	†	-	0.7	-	-	-
	C	†	-	0.3	-	1.6	-	-	-
HEAVY STEEL									
Open-Web Joist with Metal Deck	A	†	-	†	-	0.9	-	-	-
	B	†	-	0.2	-	1.4	-	-	-
	C	†	-	0.6	-	2.3	-	-	-

Note: Overpressure values assume radiation protection equal to a P_f of 100 (18 in. of earth or equivalent) superimposed on roof. Assumed density of earth = 100 pcf.

† - Required protection (P_f = 100) would cause roof to collapse.

(1) - See Table 6-3.

(2) - MILL RACE Test.

(3) - Waterways Experiment Station Test.

If roof construction is concrete, use Floor Matrix, Light Concrete Construction.

SSI 8144-7 Final Report
September 1982

Approved for public release,
Distribution unlimited

RESEARCH ON UPGRADING STRUCTURES FOR HOST AND RISK AREA SHELTERS

by

R.S. Tansley, G.J. Cuzner, and C. Wilton

for

Federal Emergency Management Agency
Washington, D.C. 20472

Contra. No. EMW-C-0686, Work Unit 1128A

FEMA REVIEW NOTICE:

This report has been reviewed in the Federal Emergency Management Agency and approved for publication. Approval does not signify that the contents necessarily reflect the views and policies of the Federal Emergency Management Agency.

Scientific Service, Inc.
517 East Bayshore, Redwood City, CA 94063

ACKNOWLEDGEMENTS

This report describes upgrading concepts on building systems designed to provide shelter from nuclear weapons effects, develops practical techniques for structural upgrading prediction, and attempts to substantiate the concepts and predictions by laboratory testing.

The authors wish to take this opportunity to thank all those involved in this project, with particular thanks to Dr. Michael A. Pachuta of the Federal Emergency Management Agency for his contributions to overall project direction and his thorough and helpful technical review.

Particular thanks for contributions to this project are due to Dr. T. C. Zsutty of San Jose State University, and to our colleagues on the SSI technical staff: Dr. B. L. Gabrielsen and Ronald D. Bernard.

Finally, the authors would like to acknowledge the support staff of SSI, Evelyn Kaplan, Larue Wilton, and Maureen Ineich for their talent and patience.

TABLE OF CONTENTS

LIST OF FIGURES

Number		Page
3-6	Photographs of 4-3/4" by 14-1/2" Glulam Beam Test Specimen Under Construction	24
3-7	Construction Details of 3-1/8" by 18" Glulam Test Specimen	25
3-8	Load Configuration and Deflection Gauge Location for the Unshored 4-3/4" by 14-1/2" Glulam Beam	27
3-9	Duration of Load Adjustment Factors to Correct for Loads Other Than of 10-Years Duration	28
3-10	Load vs Midspan Deflection of the Base Case Test Specimen	29
3-11	Load, Shear, and Moment Relationships for the Base Case Test Beam at Failure	30
3-12	Pre- and Post-Test Photographs of 4-3/4" by 14-1/2" Glulam Base Case Test Specimen	31
3-13	Closeup of Failed Finger Joint and Area of Failure	32
3-14	Load Configuration and Deflection Gauge Locations for the Shored at Midspan Glulam Beam Tests	34
3-15	Load vs Midspan Deflection for the Shored at Midspan Test Specimen	35
3-16	Load, Shear, and Moment Relationships for the Failed Glulam	36
3-17	Plot of Bearing Test Results for a Section of the Failed Beam	37
3-18	Midspan Shore With 10-Inch Long Bearing Plate, Showing Bearing/Shear Failure	38
3-19	Failed and Unfailed Bearing Surfaces at Midspan	39
3-20	Load vs Midspan Shore Reaction for the Shored at Midspan Test Specimen	41
3-21	Load, Shear, and Moment Relationships for the Failed Glulam Beam	42
3-22	Load, Shear, and Moment Relationships for the Glulam Beam That Did Not Fail	43
3-23	Pretest Photographs of 4-3/4" by 14-1/2" Glulam Beam Test Specimen Shored at Midspan With a 12-Inch Long Bearing Plate	44

Number		Page

Number		Page
3-40	Pre- and Post-Test Photographs of Test Specimen	64
3-41	Posttest Photographs of Test Specimen	65
3-42	Flexural-Finger Joint Failure in the Tension Laminate on the Failed Beam	66
4-1	Connections for One- and Two-Way Slabs	73
4-2	One-Way Reinforced Concrete Joists	75
4-3	Flat Slab Connections	76
4-4	Flat Plate Connections	78
4-5	Waffle Slabs	80
4-6	Post-Tensioned Connections	82
4-7	Post-Tensioned Connections	84
4-8	Floor and Roof Slabs	87
4-9	Hollow-Core Slab Connections (Untopped)	90
4-10	Hollow-Core Slab Connections (Topped)	91
4-11	Hollow-Core Slab Connections (Topped)	92
4-12	Double Tee Connections (Untopped)	93
4-13	Double and Single Tee Connections (Topped)	94
4-14	Double and Single Tee Connections (Topped)	95
4-15	Girder Shapes	100
4-16	Girder Connections	101
4-17	Precast Column Base Connections	103
4-18	Precast Column to Column Connections	105
4-19	Reinforced Concrete Cast-in-Place Column Base Connections	106
4-20	Wall to Floor or Roof Connections	110

LIST OF TABLES

Section 1
INTRODUCTION

Current Civil Defense planning in the United States is based on a policy of "Crisis Relocation", which assumes that a period of crisis buildup or international tension will allow time — a few days or weeks — to evacuate approximately 80% of the population to host areas. Ultimate survival of the evacuated population and of the key workers who will remain behind to maintain essential industries and services depends on the provision of adequate shelters to protect them from the blast and radiation effects of nuclear weapon attack. There does not exist, in either the host or the risk areas, sufficient shelter space. Therefore, a major facet of preparedness is the upgrading of existing structures and the provision of expedient shelters to provide shelter and protection.

To provide the engineering basis and guidance for the development of these shelters Scientific Service, Inc., is conducting a five-year program of research on upgrading structures for host and risk area shelters. This report presents a summary of the work conducted during the first year of this five-year program.

This report is organized as follows: Section 2 - static and dynamic tests of prestressed precast hollow-core floor planks; Section 3 - glulam beam tests; Section 4 - review of the types of concrete connections and connection systems that most directly affect the performance of potential shelter options; Section 5 - analysis of the punching strength of reinforced concrete roof and floor slabs; and Section 6 - program summary cnd Conclusions. There are four appendixes to the report: Appendix A presents example upgraded buildings; Appendix B, a prediction methodology; Appendix C, calculations for the glulam beam tests; and Appendix D, glulam analysis.

Section 2
STATIC/DYNAMIC FLOOR TESTS

INTRODUCTION

During last year's program, a number of prestressed precast hollow-core planks of varying thicknesses were load tested hydraulically in the laboratory and reported in SSI report No. 8012-6 (Ref. 1). The purpose of these tests was to determine the load-carrying capacity of the elements, shored and unshored, and to develop a failure prediction methodology.

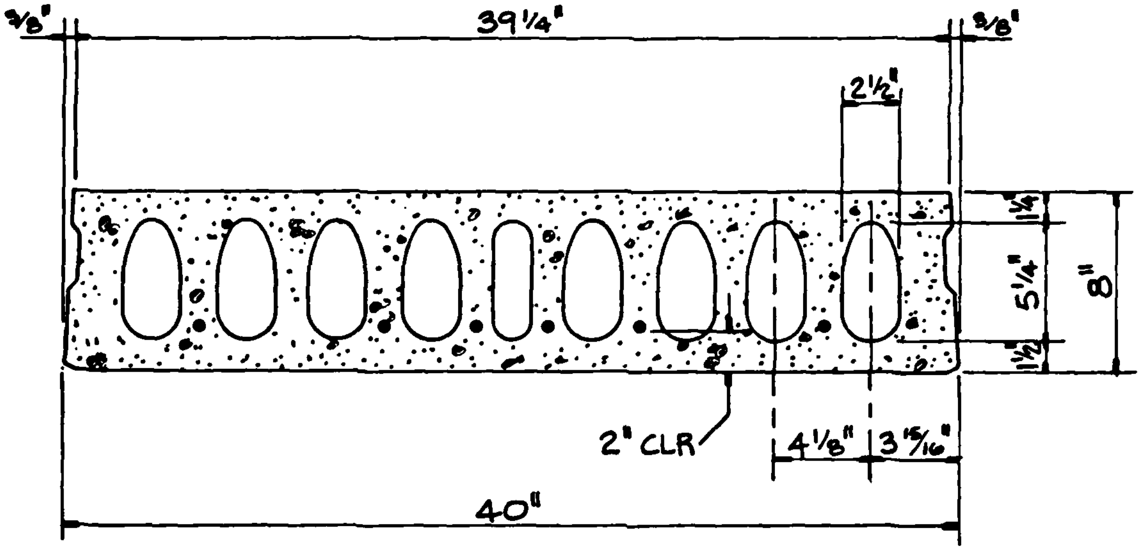
The overall goal of this test program is to evaluate the performance of these building elements under blast loading. Accordingly, dynamic load tests on identical elements were conducted to verify that the dynamic response was comparable to that of the previously conducted static tests. The test methods employed were developed through a small-scale program and were previously applied to full-scale wood floor tests (Ref. 2).

TEST ELEMENTS AND PROCEDURES

Three tests were conducted on 8-inch thick prestressed precast concrete planks. The plank cross section and properties, shown on Figure 2-1, are identical to the plank tested and reported in Ref. 1.

All three tests had the same test configuration and, as shown on Figure 2-2, all were conducted in the same manner with the same loading weights. A 14,000-lb crane counterweight was used for the preload, applying a load of 7,000 lb to each one-third point on the plank. The drop weight was a 2,500-lb concrete block. The drop weight was suspended from a crane hook a calculated distance above the preload weight, and then released to free fall by use of a quick release mechanism attached to the crane hook.

8" PRESTRESSED PRECAST HOLLOW-CORE PLANK



SECTION PROPERTIES

$A = 218\ in^2$	$I = 1515\ in^4$
$Y_T = 4.02\ in$	$Y_B = 3.98\ in$
$S_T = 377\ in^3$	$S_B = 380\ in^3$

DESIGN SPAN = 18 ft 0 in
f'_c = 5000 PSI
WGT = 60 lbs/ft^2
STRANDS = 6 - 3/8 in dia - 270^K
MEDIUM DESIGN LOAD = 165 lbs/ft^2

Fig. 2-1. Design Properties of 8-inch Prestressed Precast Hollow-Core Plank.

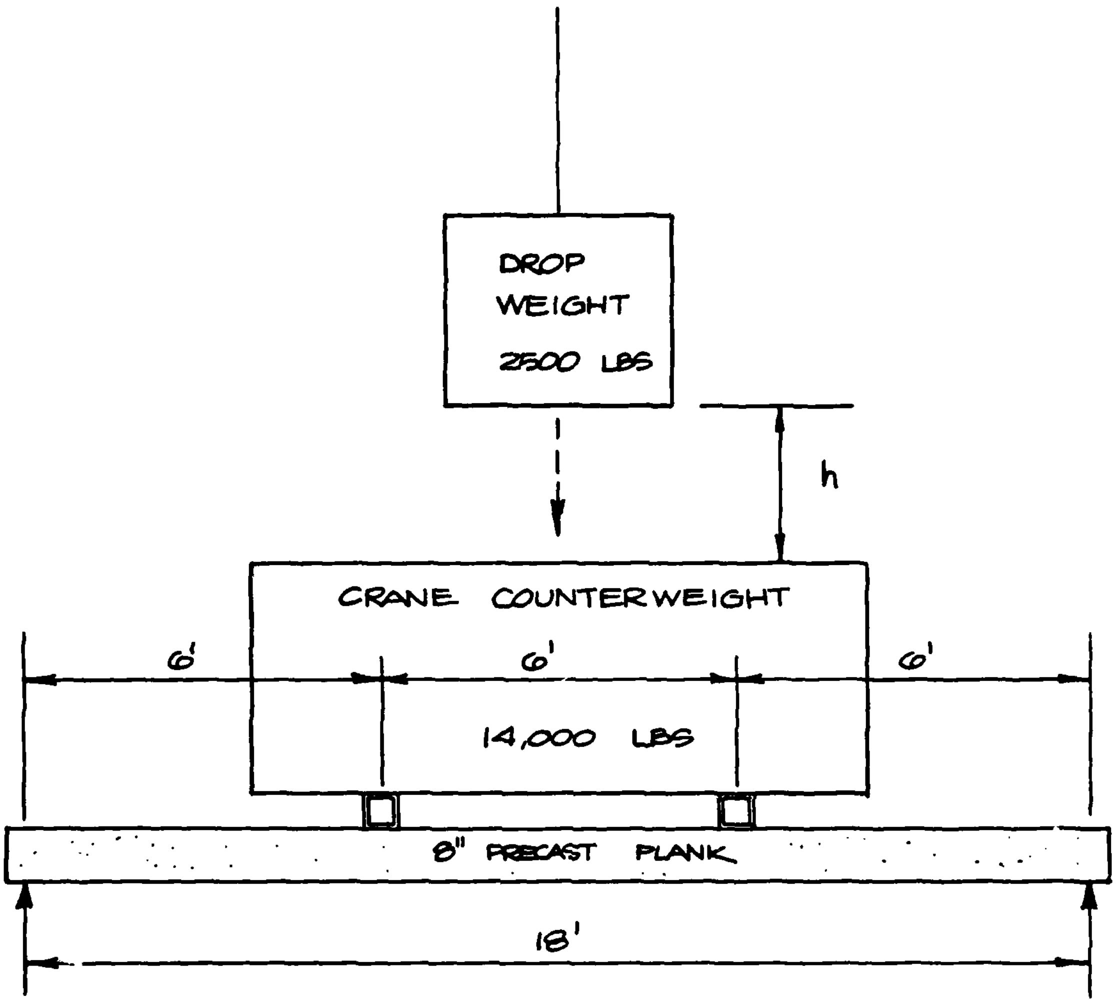


Fig. 2-2. Test Configuration.

PREDICTION METHOD

The method used to predict failure of the concrete plank is based on the energy required to fail identical planks in the static tests. The analysis used was based on determining the total energy required; i.e., the area under the load-deflection curve of planks previously statically tested to failure, Figure 2-3. By applying a 14,000-lb preload (7,000 lb at each one-third point) and measuring the midspan deflection, the amount of energy remaining, and thus that required to cause failure, may be estimated. Referring to Figure 2-3, with the preload applied, the total energy remaining is theoretically the portion of the load-deflection curve above the preload level less any energy losses. The calculations for the strain-energy remaining in the plank after preload for the three tests are attached as Table 2-1. These calculations were based on using the 2,500-lb drop weight previously described.

A graphic presentation of the calculations of strain energy is shown on Figure 2-4, a more detailed version of the upper portion of Figure 2-3, for each of the tests. In order to calculate the drop height, the efficiency of energy applied to induce failure must be determined. The efficiency selected for the first test (25%) was based on the small-scale experiments reported in Ref. 2. After experience was gained from the first test, a better estimate of energy efficiency could be made, and in succeeding tests, this value was reduced to 20%. The 80% loss in energy was due to thermal energy, in the form of heat, as well as sound and the work performed in the distortion and deflection of the test apparatus and components.

TEST RESULTS AND CONCLUSIONS

Results of the tests are listed in Table 2-2 for each of the three planks tested. Photographs of the apparatus used, and the failures that resulted, are shown in Figures 2-5 through 2-12.

The results verified that dynamic load tests to failure could be predicted using the concept of conservation of energy in conjunction with the load-deflection curves obtained from static tests on identical specimens.

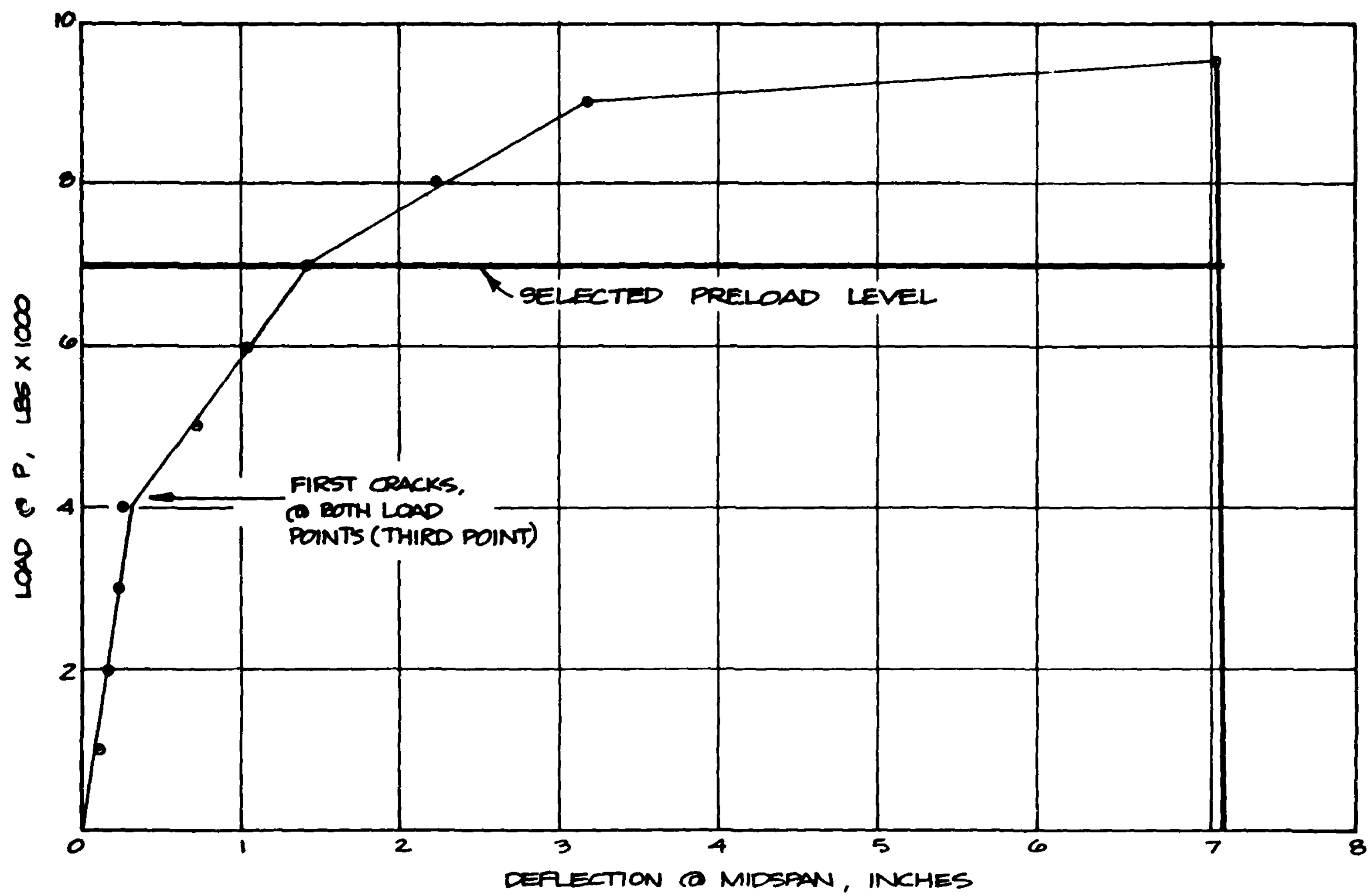


Fig. 2-3. Load Deflection Curve.

TABLE 2-1: CALCULATIONS

1. CALCULATION OF STRAIN ENERGY REMAINING IN SLABS AFTER PRELOAD.

$$U = \text{AREA } (1) + \text{AREA } (2)$$

$$U = \left[\frac{1.8 \times 2000}{2} + \frac{4\,(2000 + 2500)}{2}\right] 2/12$$

$$U = \underline{1800 \text{ FT-LB}}$$

2. TEST 1 - ASSUME 25% EFFICIENCY WITH 2500 LB DROP WEIGHT, USE STATIC CURVE. DETERMINE DROP HEIGHT, "h".

$$h = \frac{1800}{0.25 \times 2500} = 2.88 \text{ FT.} = \underline{34 \text{ IN.}}$$

SLAB DID NOT FAIL, TWO ADDITIONAL DROPS FROM 18 IN. AND 24 IN. DID NOT CAUSE FAILURE - TEST ABANDONED.

3. TEST 2 - ASSUME 20% EFFICIENCY WITH 2500 LB DROP WEIGHT. CORRECT AREA UNDER CURVE FOR ACTUAL DEFLECTION UNDER PRELOAD, 1.6 IN.

$$U = \left[\frac{1.6 \times 2000}{2} + \frac{4\,(2000 + 2500)}{2}\right] 2/12 = 1767 \text{ FT-LB}$$

$$h = \frac{1767}{0.20 \times 2500} = 3.53 \text{ FT.} = \underline{42 \text{ IN.}}$$

4. TEST 3 - ASSUME 20% EFFICIENCY WITH 2500 LB DROP WEIGHT. CORRECT FOR 0.2 IN. DEFLECTION UNDER PRELOAD.

$$U = \left[\frac{3 \times 2000}{2} + \frac{4\,(2000 + 2500)}{2}\right] 2/12 = 2000 \text{ FT-LB}$$

$$h = \frac{2000}{0.20 \times 2500} = 4.0 \text{ FT.} = \underline{48 \text{ IN.}}$$

INCREASE "h" 6 IN., 48 + 6 = $\underline{54 \text{ IN.}}$

DETERMINE ACTUAL EFFICIENCY

$$\text{ENG. EFF} = \frac{2000}{4.5 \times 2500} = \underline{18\%}$$

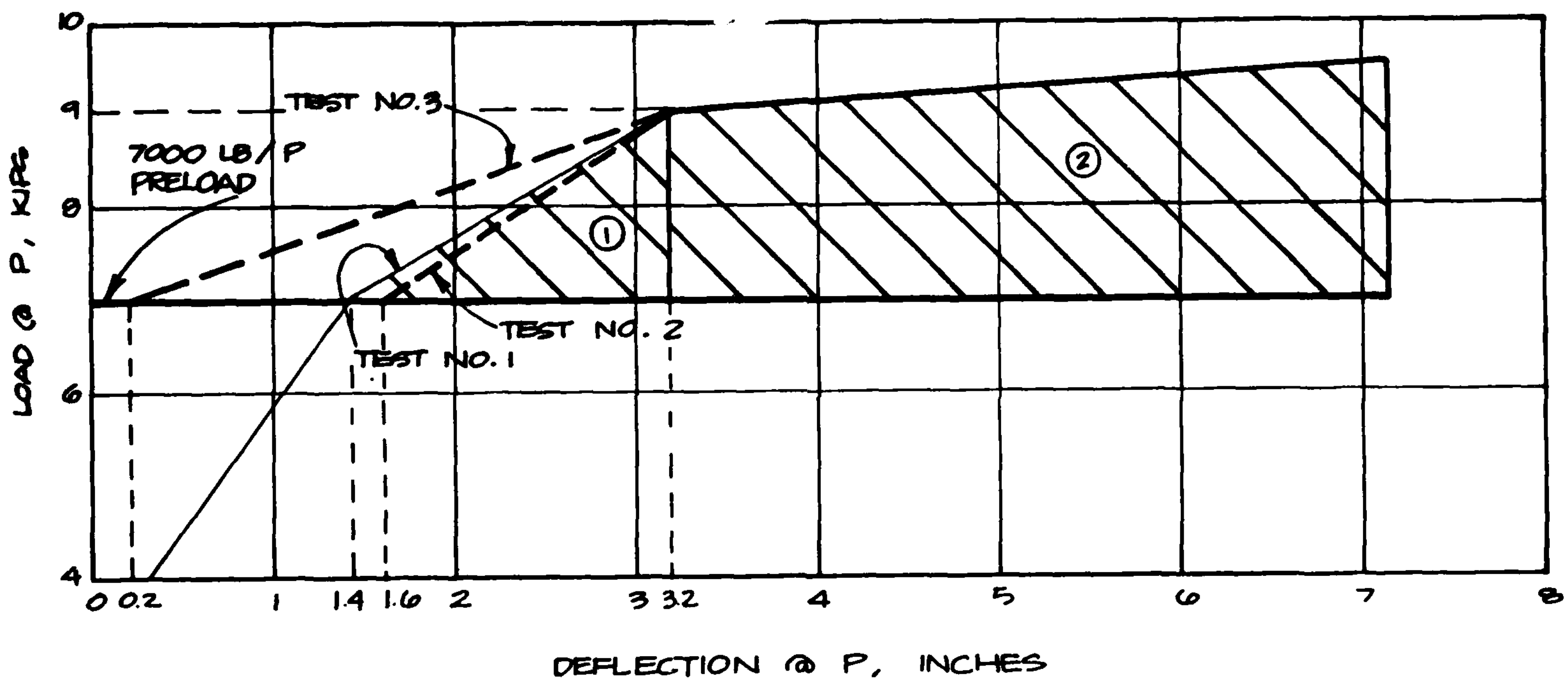


Fig. 2-4. Strain Energy Calculations After Preload, Tests 1 Through 3.

TABLE 2-2: DROP TEST DATA

TEST 1

Deflection under preload	1.06 inches (27 mm)
Energy efficiency (estimated)	25 percent
Drop distance (calculated)	34 inches (864 mm)
Results (no failure)	3.66 inches (93 mm) deflection
Drop distance (calculated)	18 inches (457 mm)
Results (no failure)	4.76 inches (121 mm) deflection
Drop distance (calculated)	24 inches (110 mm)
Results (partial failure)	No further tests

TEST 2

Deflection under preload	1.61 inches (41 mm)
Energy efficiency (estimated)	20 percent
Drop distance (calculated)	42 inches (1067 mm)
Results (slab failed)	

TEST 3

Deflection under preload	0.2 inches (6 mm)
Energy efficiency (estimated)	20 percent
Drop distance	54 inches (1372 mm)
Results (slab failed)	

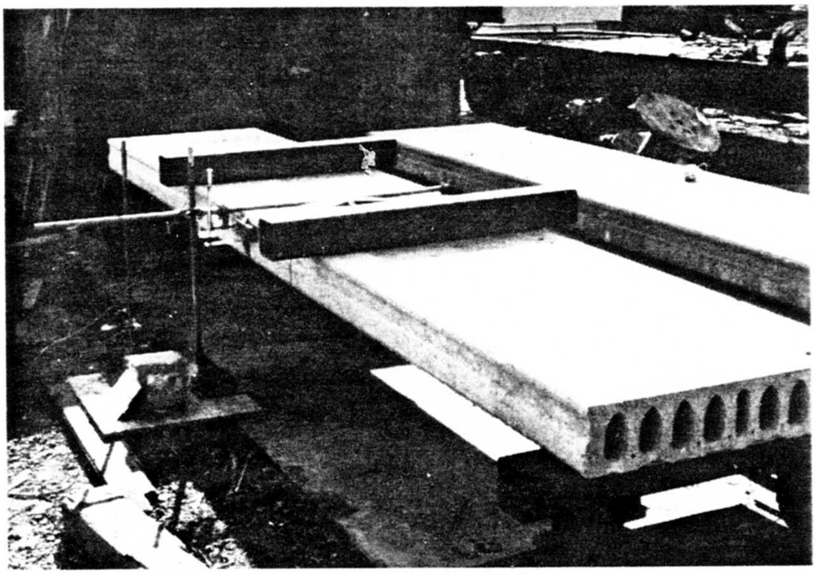


Fig. 2-5. Test No. 1, Plank Prior to Preload Application.

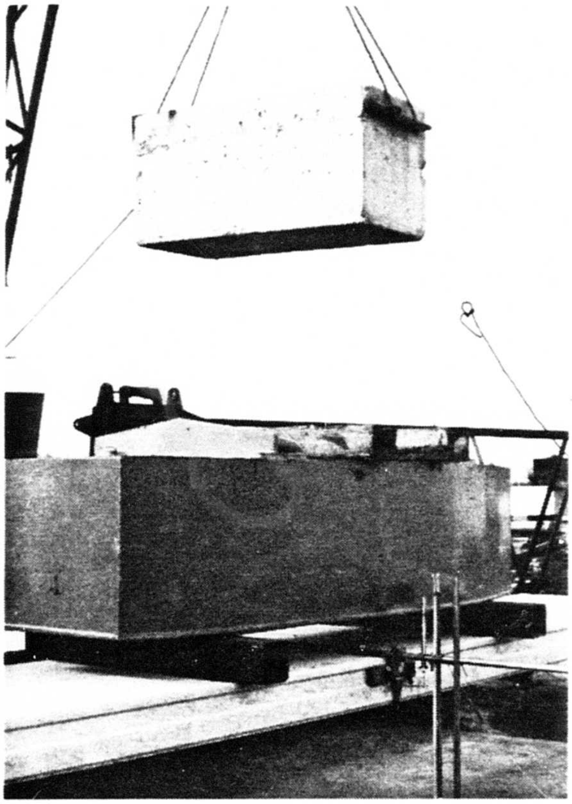


Fig. 2-6. Test No. 1, Plank With Preload and Drop Weight in Position.

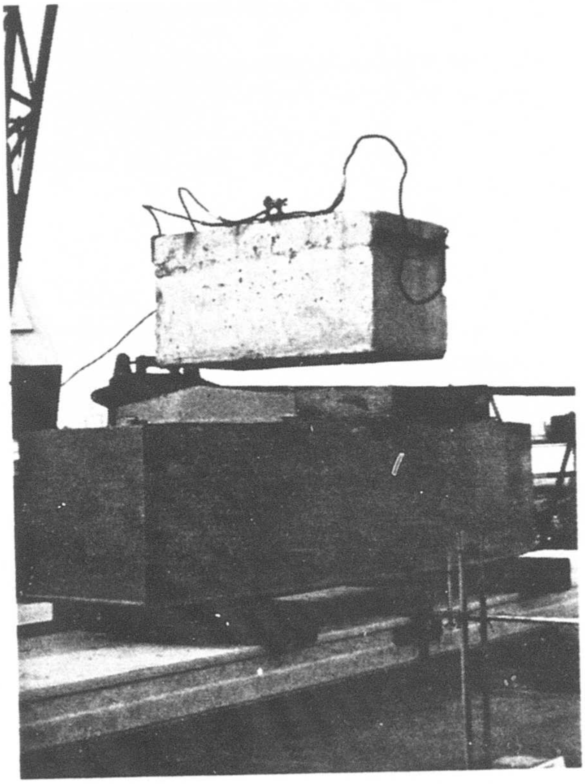


Fig. 2-7. Test No. 1, Drop Weight in Free Fall.

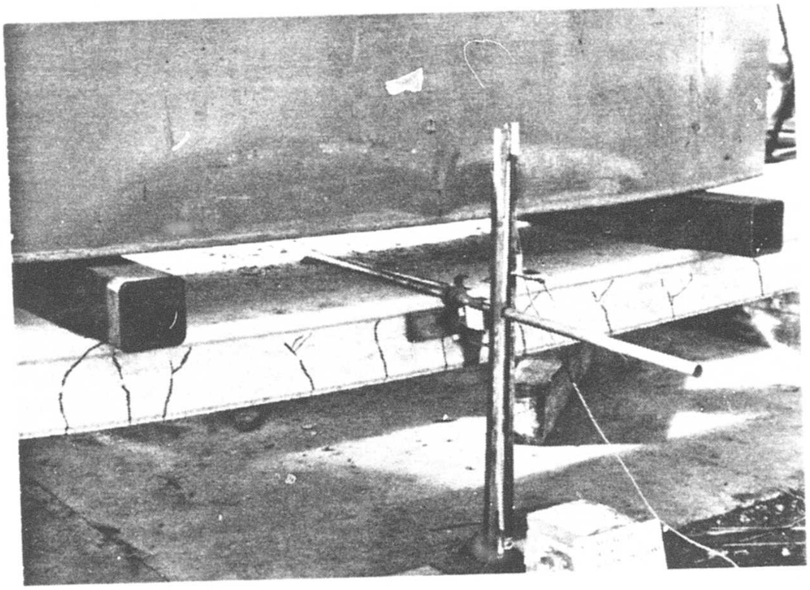


Fig. 2-8. Test No. 1, Partial Slab Failure After Final Drop - No Collapse.

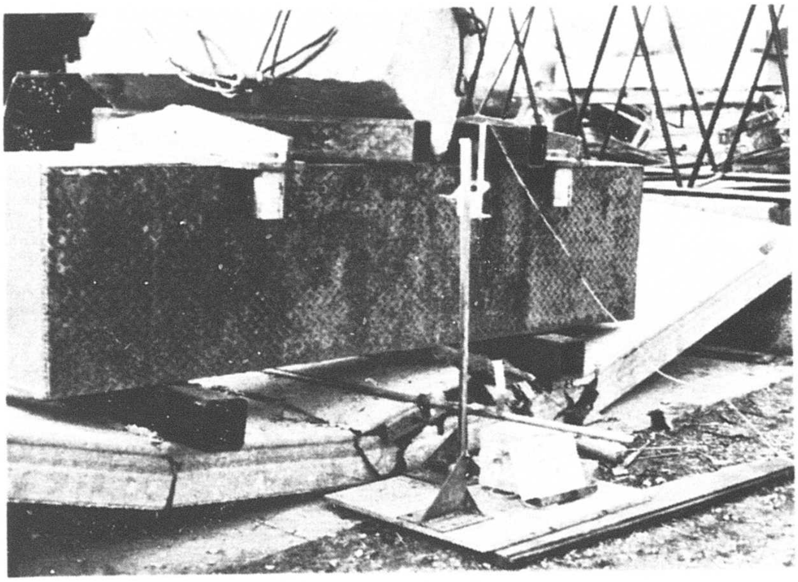


Fig. 2-9. Test No. 2, Showing Plank Failure.

Fig. 2-10. Plank Failure After Load Was Removed.

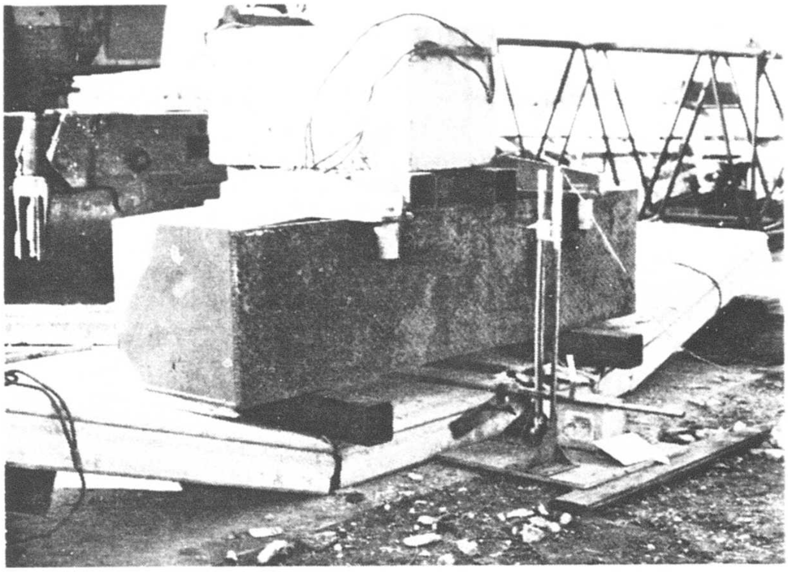


Fig. 2-11. Test No. 3, Showing Plank Failure.

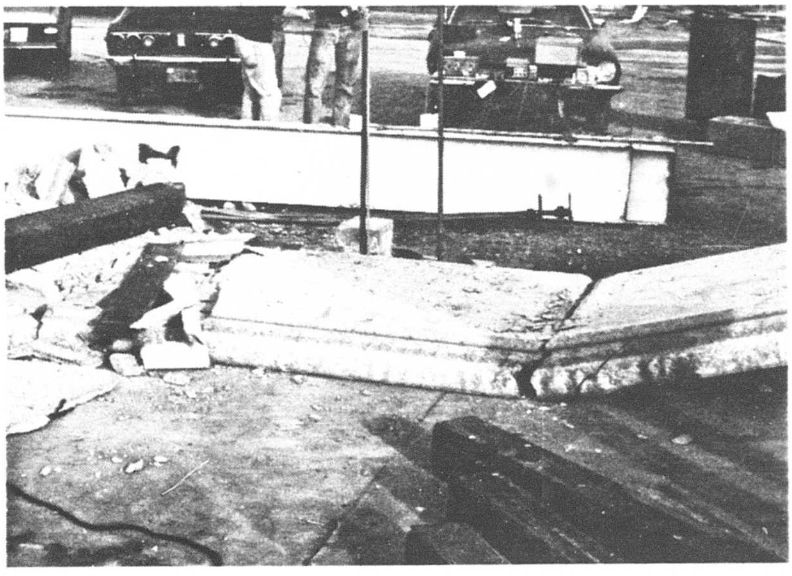


Fig. 2-12. Test No. 3, Plank Failure After Load Was Removed.

Section 3
GLULAM BEAM TESTS

INTRODUCTION

Glulams are widely used throughout the construction industry, and their applications vary. For example, they are used in motels, offices, theaters, warehouses, retail stores, and manufacturing and commercial buildings, all structures that are planned to be used as host area shelters.

The objective of this program was to determine the load-carrying capacity of typical floor and roof glulams, both when unshored and when shored at midspan. The glulams selected for testing were: (1) a cross section 6-3/4" wide by 16-1/2" deep, typically used in floor systems, (2) a cross section 3-1/8" wide by 18" deep, typically used in roof systems. This second glulam was specifically selected because of its large depth-to-thickness ratio of 6, to determine if lateral stability would be a problem with midspan shoring.

The glulam beams were obtained from a local supplier and were fabricated in accordance with Ref. 3 and Ref. 4. The 6-3/4" by 16-1/2" glulams were built up using 1-1/2" by 6-3/4" by 24'0" douglas fir or larch laminates. A total of 11 laminates were used, and finger joints were used to join sections of laminates to form the single pieces 24' long. A photograph of one of these finger joints is presented in Figure 3-1. The 3-1/8" by 18" by 24'0" glulams were built up of 1-1/2" by 3-1/8" by 24'0" douglas fir or larch laminates. Twelve laminates were used.

Beam Framing System Design

To assist in the design of the test specimens typical floor and roof designs using these glulam beams were developed. The floor system using the 6-3/4" wide glulams is shown in Figure 3-2. In this design the glulams would be spaced 12'0" on center, and solid sawn purlins, 4" by 12" by 12'0", would be framed on to the glulams

8'0" on center using purlin hangers. Subpurlins, 2" by 8" by 8'0", would be framed onto the 4" by 12" purlins at 16" on center, and the entire assembly sheathed with 3/4" plywood. The roof system using the 3-1/8" wide glulams is shown in Figure 3-3. In this design the glulams would be spaced 16'0" on center and 4" by 12" purlins installed at 8'0" on center. Stiffeners, 2" by 4" subpurlins, would be installed at 16" on center. Structural calculations supporting each of these designs are presented in Appendix C.

Test Specimen Construction

The test specimens were built to represent the floor and roof systems noted above. Since the intent of the tests was to determine the failure strength of the glulam beams, as-built and upgraded, and not to test the entire floor and roof systems, the spacing between the beams was reduced. To provide stability, the purlins and plywood specified in the designs were installed on the glulam beams.

Each test specimen consisted of two 24'0" long glulam beams located 24" on center and either 4" by 10" or 4" by 12" purlins were framed onto the glulams at 8' on center. Finally, plywood sheathing was nailed on top of the assemblies in conformance with the minimum nailing schedule of the 1979 UBC (Ref. 5). Details of the 6-3/4" by 16-1/2" glulam beam test specimens are shown in Figure 3-4. Construction photographs of one of these test specimens are shown in Figures 3-5 and 3-6. Details of the 3-1/8" by 18" test specimens are shown in Figure 3-7.

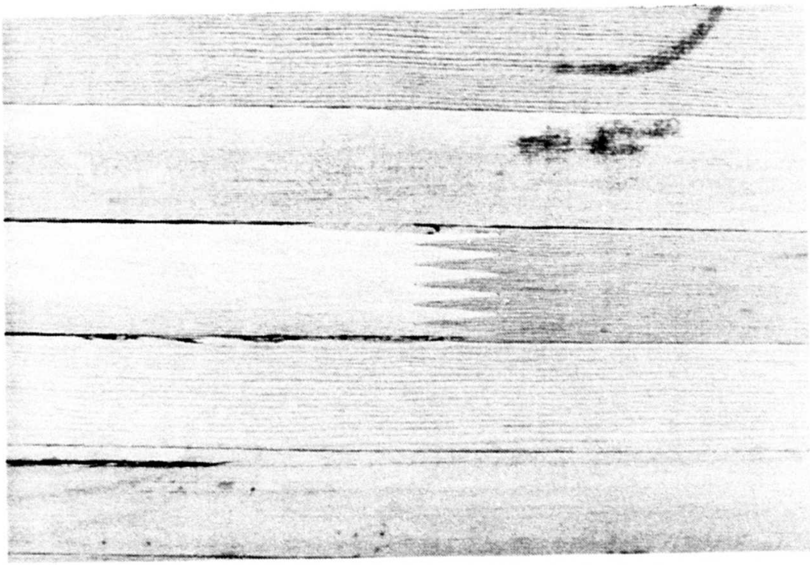
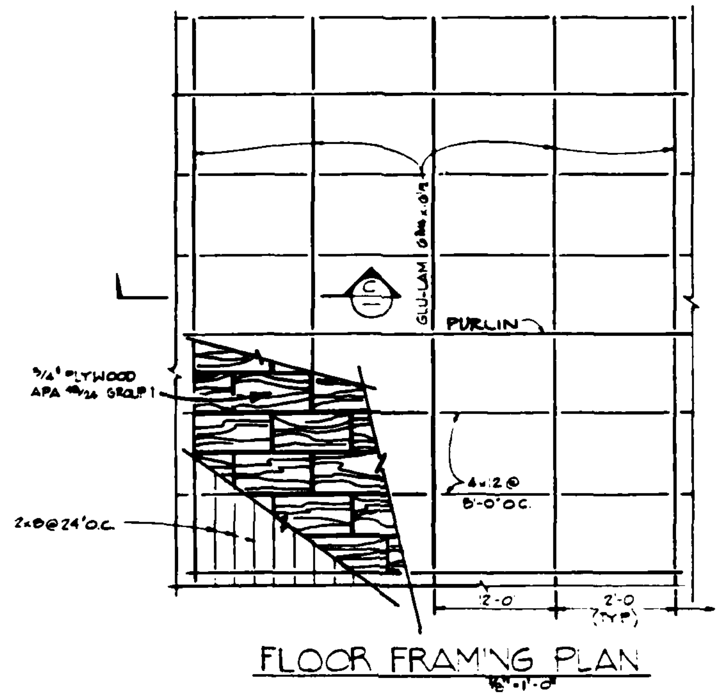


Fig. 3-1. Example of Finger Joint Used in the 6-3/4" by 16-1/2" Glulams.



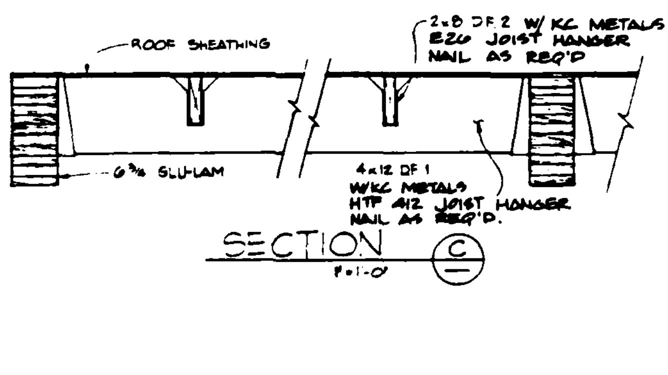


Fig. 3-2. Framing Plan for Floor System Using 6-3/4" Wide by 16-1/2" Deep Glulams.

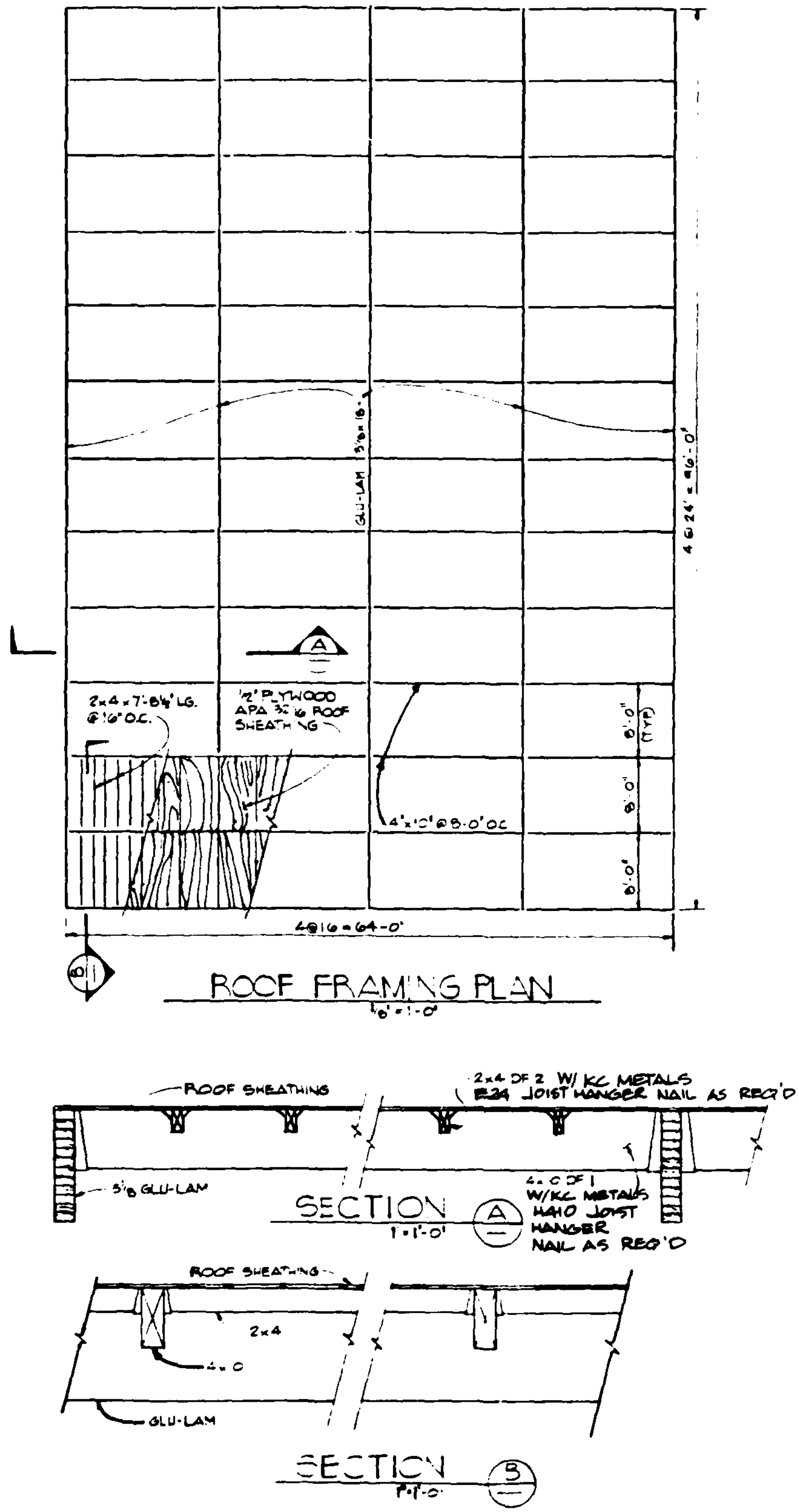


Fig. 3-3. Framing Plan for Roof System Using 3-1/8" Wide by 18" Deep Glulams.

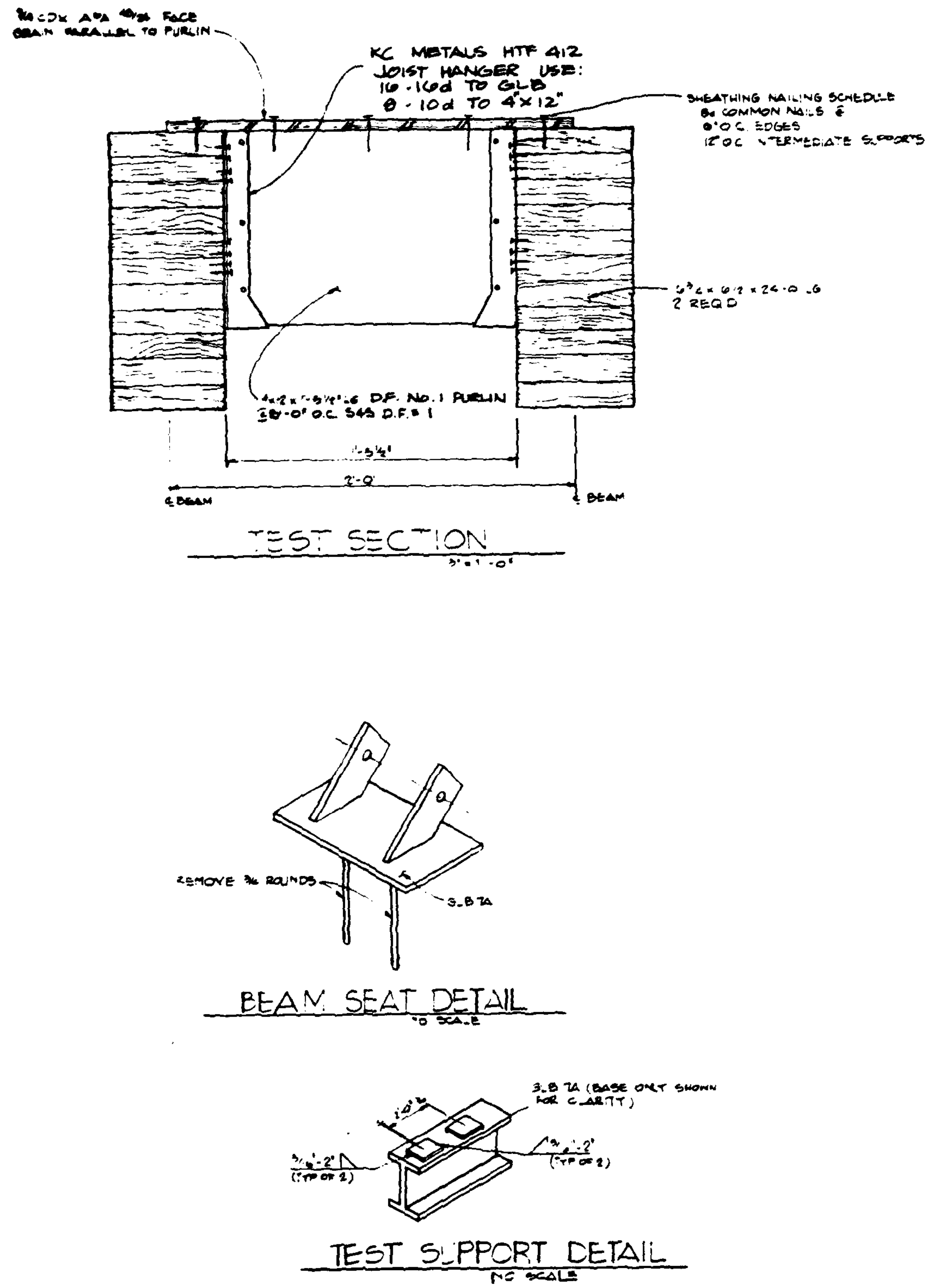


Fig. 3-4. Construction Details of 6-3/4" by 16-1/2" Glulam Beam Test Specimens.

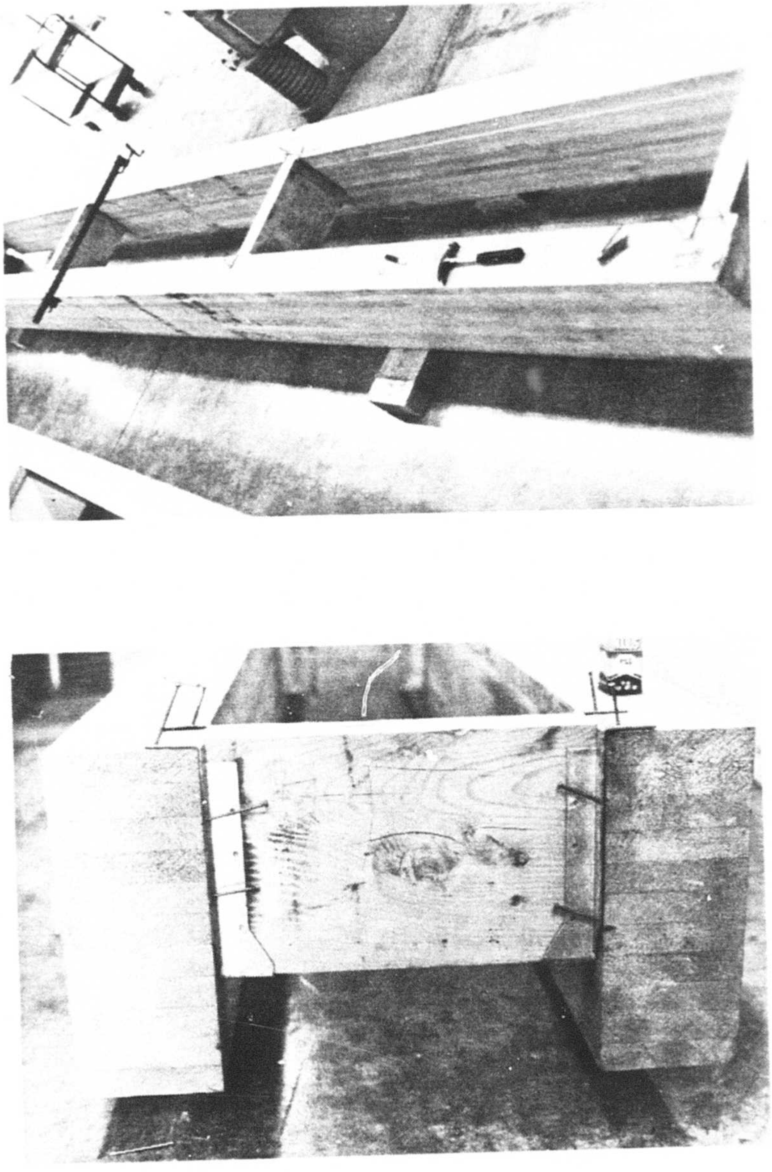
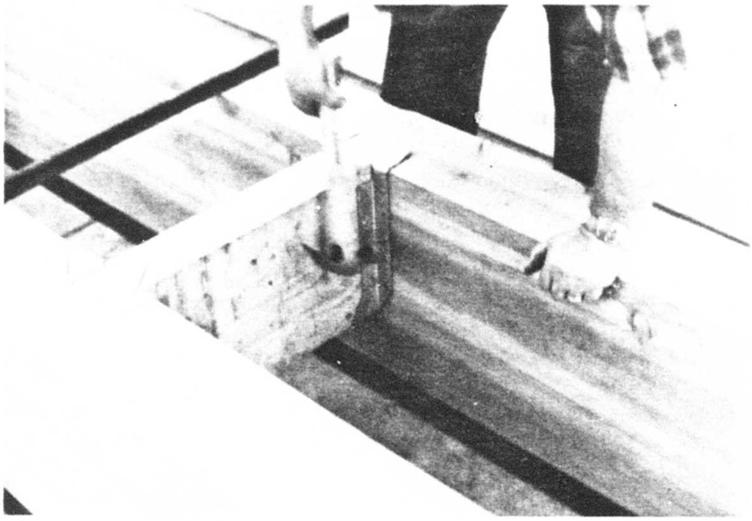


Fig. 3-5. Photographs of 6-3/4" by 16-1/2" Glulam Beam Test Specimen Under Construction.



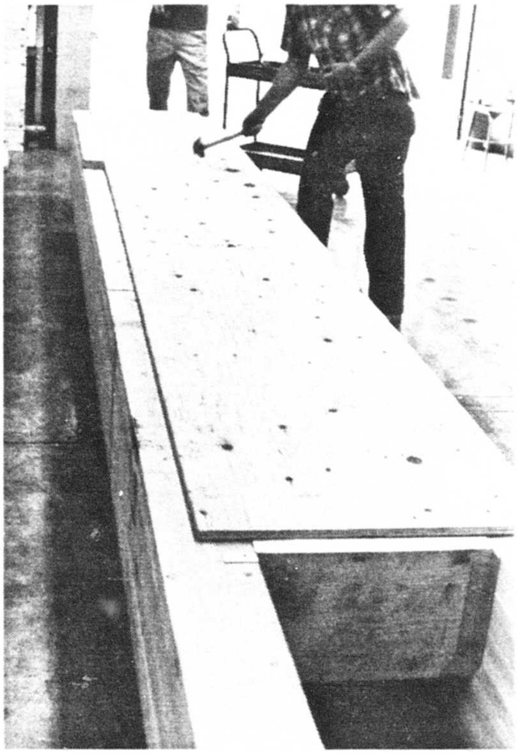


Fig. 3-6. Photographs of 6-3/4" by 16-1/2" Glulam Beam Test Specimen Under Construction.

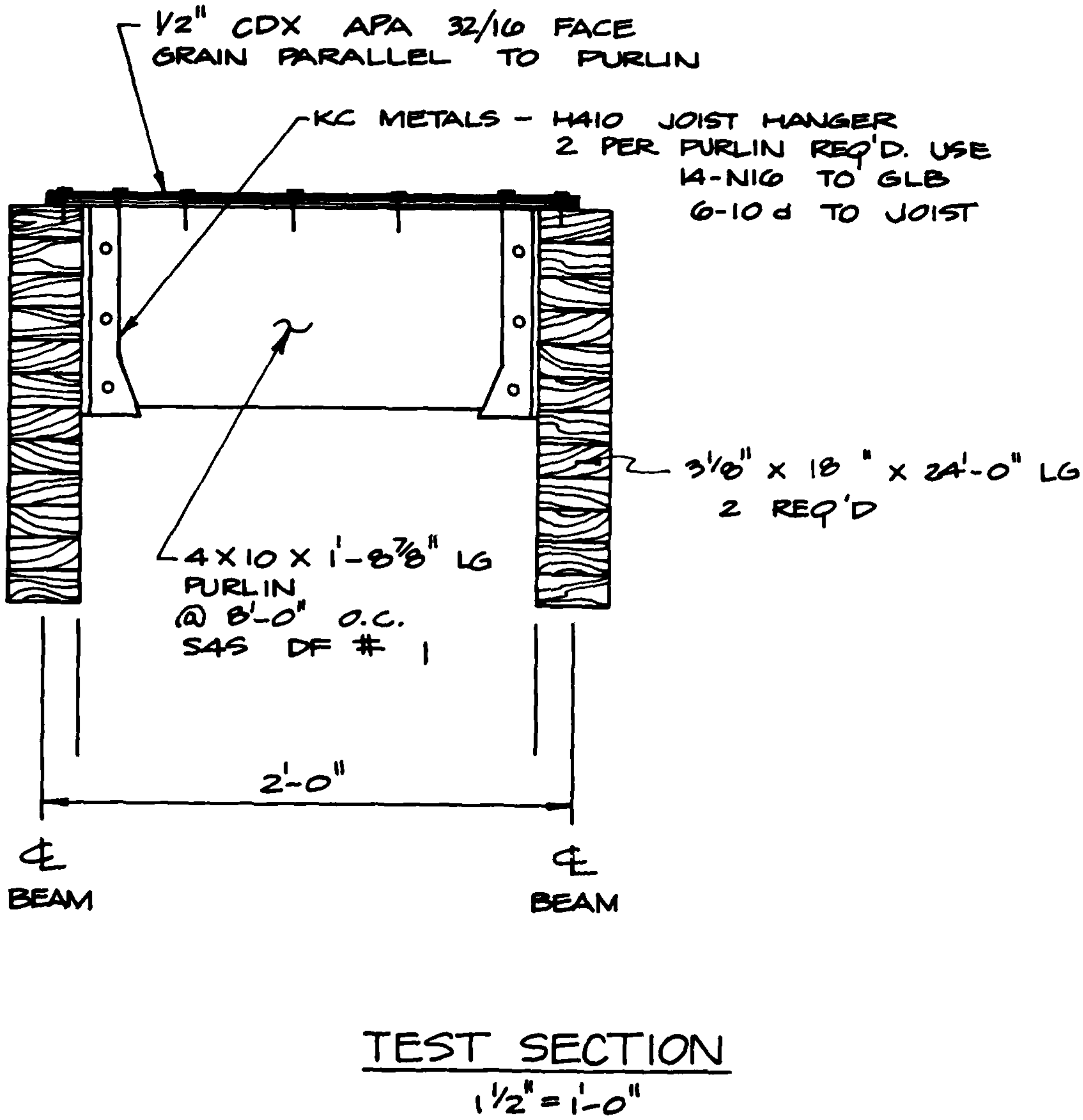


Fig. 3-7. Construction Details of 3-1/8" by 18" Glulam Test Specimen.

TEST SERIES PERFORMED ON 6-3/4" x 16-1/2" GLULAM SECTION

Unshored Base Case

The base case was conducted on an assembly of two unshored 6-3/4" x 16-1/2" x 24'0" long simply supported glulam beams. The test was performed to determine the load required to fail the test specimen and to establish a basis for comparison of upgrading methods. The ends of the test specimen were supported by a 5" x 9" beam seat. The loading arrangement and the deflection gauge locations for this test are shown on Figure 3-8.

Load was stepped slowly to failure in 2,000 lb per ram increments. Failure occurred when the average applied load was 33,000 lb per ram, or an equivalent uniform load of 1,833 plf per beam. The total combined dead plus live load that resulted in failure was 1,872 plf. The test was performed over a 1½-hour total time interval; the resulting duration of load adjustment factor is 1.45 (see Figure 3-9). Normalizing the load to a 10-year duration factor of 1.0 reduces the load at failure to 1,291 plf. The design load for the beam, based upon a normal 10-year load duration, is 855 plf. Thus, the factor of safety against failure for the tested beam is 1.51. The calculated uniform load vs midspan deflection curve is shown in Figure 3-10. Figure 3-11 presents the shear and moment diagrams for the glulams at failure.

Failure was caused by flexure and occurred approximately 4 feet from midspan, in the peak positive moment zone of one beam. The other glulam beam exhibited no evidence of failure and returned to its pretest position when the load was removed. A finger joint in the bottom tension laminate of the failed glulam beam failed in the zone of maximum positive moment. The failure resulted in nearly horizontal cracks propagating in both directions over a distance of 12 feet. Figure 3-12 shows the test specimen prior to the test and the test specimen after failure. Figure 3-13 shows the failed finger joint that caused failure.

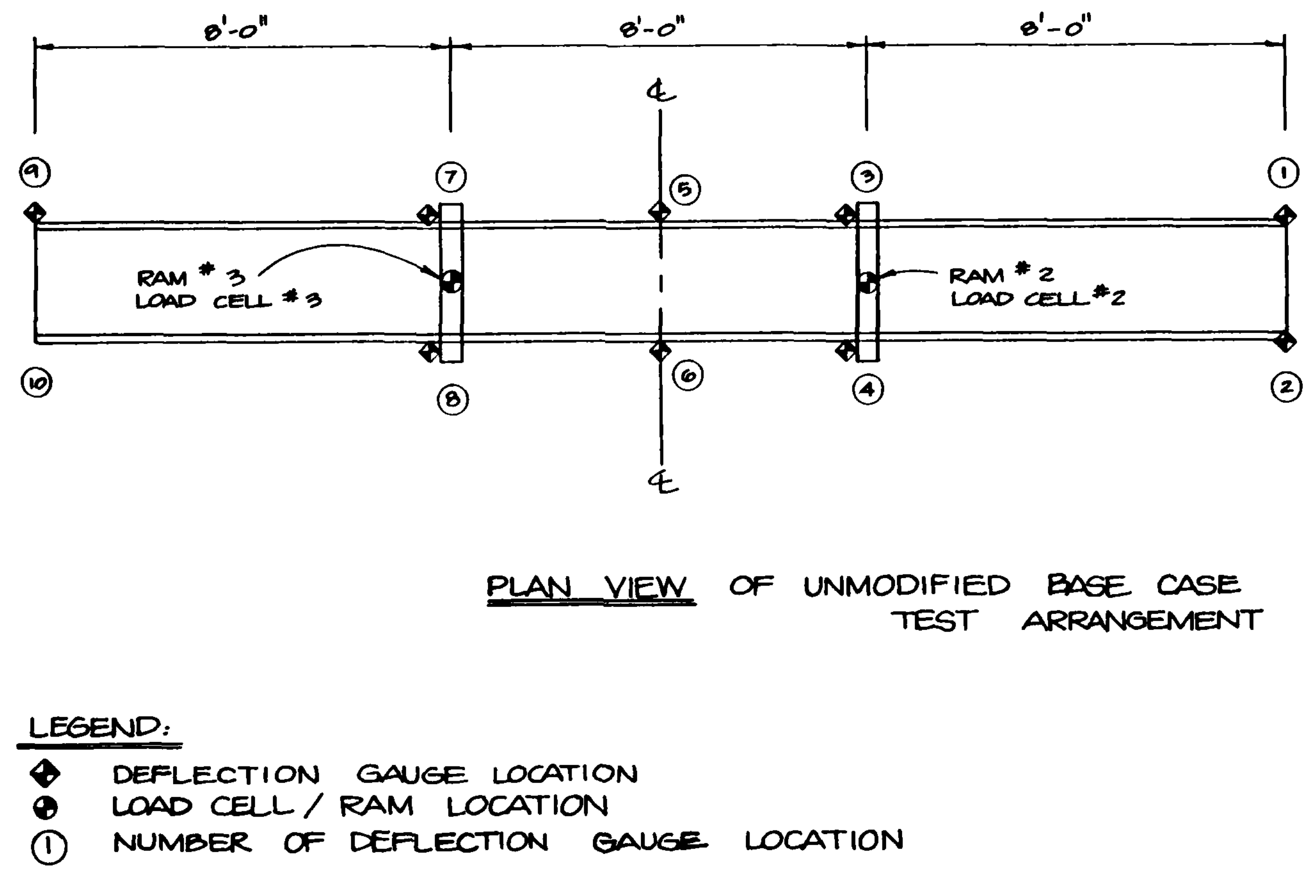


Fig. 3-8. Load Configuration and Deflection Gauge Location for the Unshored 6-3/4" by 16-1/2 Glulam Beam.

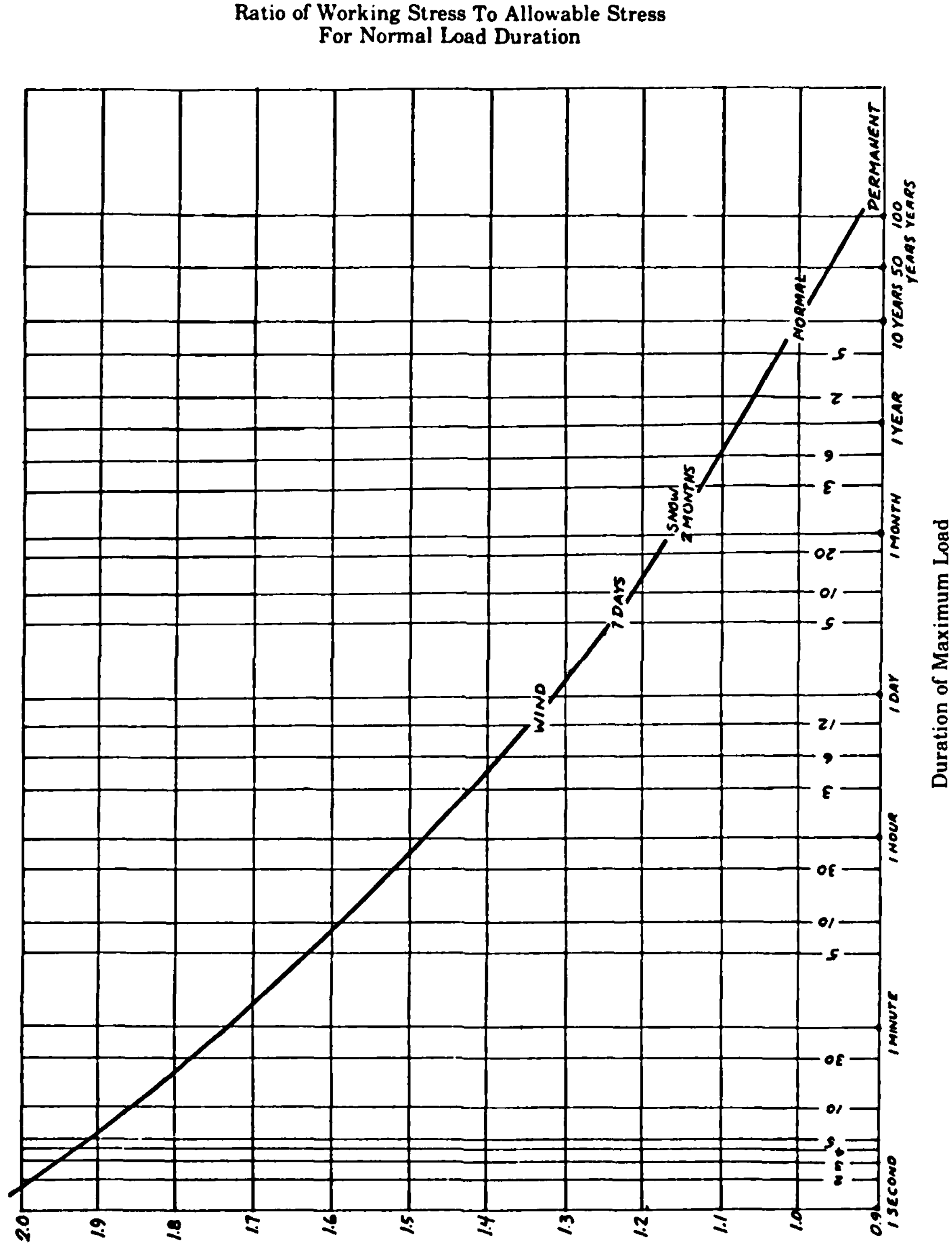


Fig. 3-9. Duration of Load Adjustment Factors to Correct for Loads Other Than of 10-Years Duration.

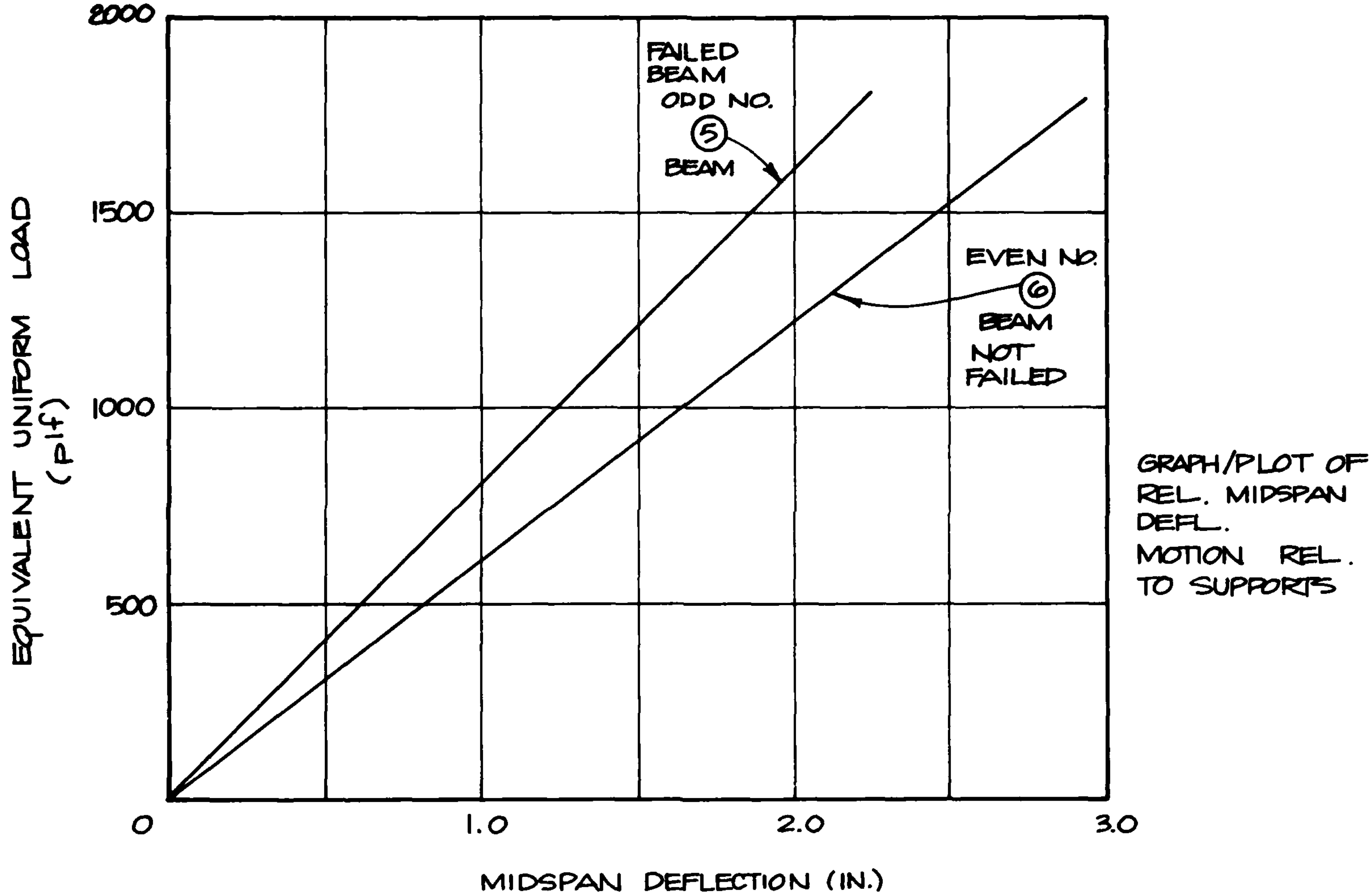


Fig. 3-10. Load vs Midspan Deflection of the Base Case Test Specimen.

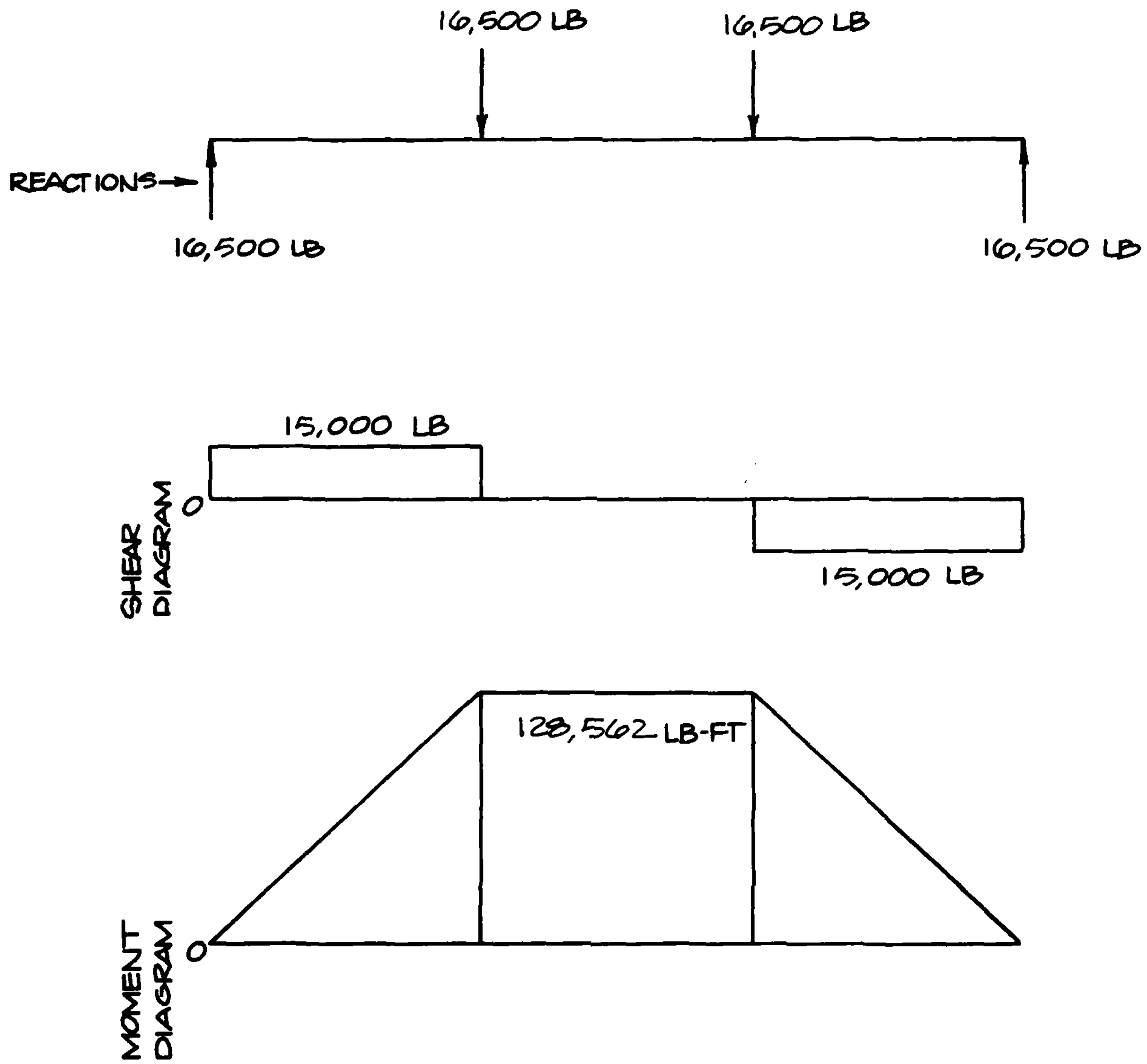
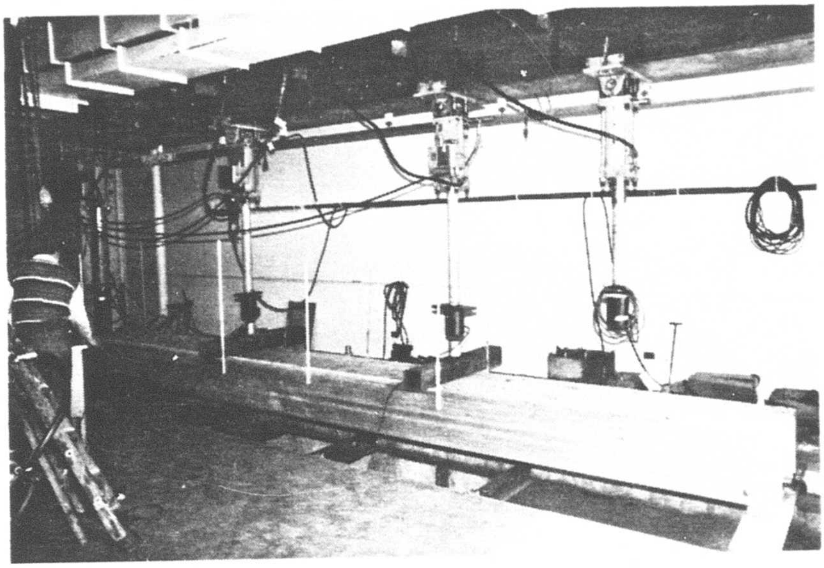


Fig. 3-11. Load, Shear, and Moment Relationships for the Base Case Test Beam at Failure.



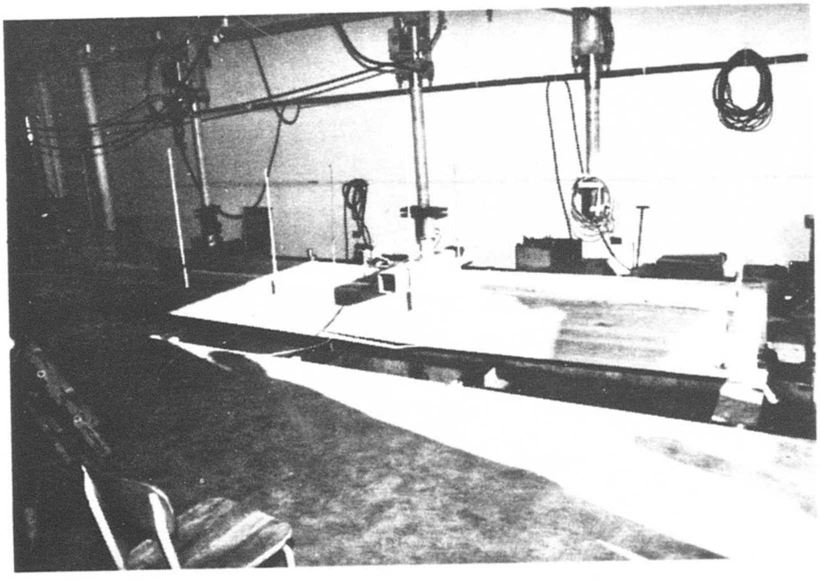
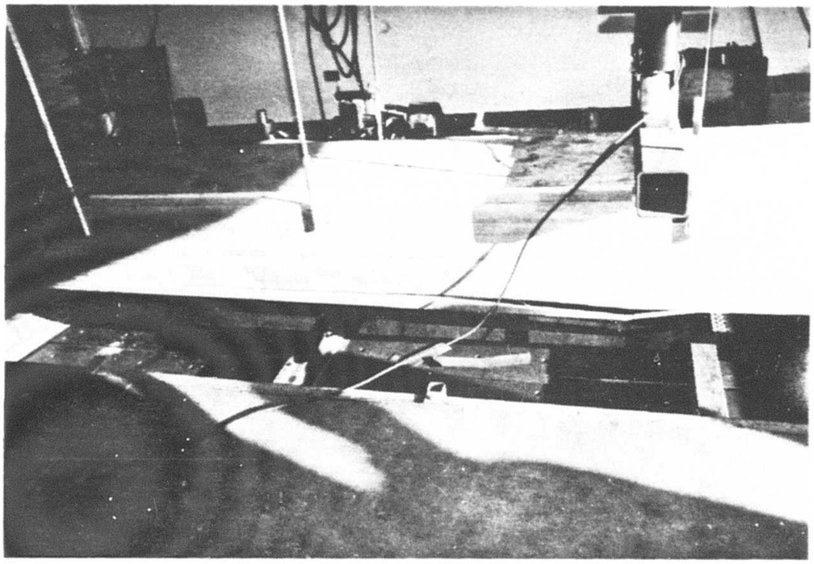


Fig. 3-12. Pre- and Post-Test Photographs of 6-3/4" by 16-1/2" Glulam Base Case Test Specimen.



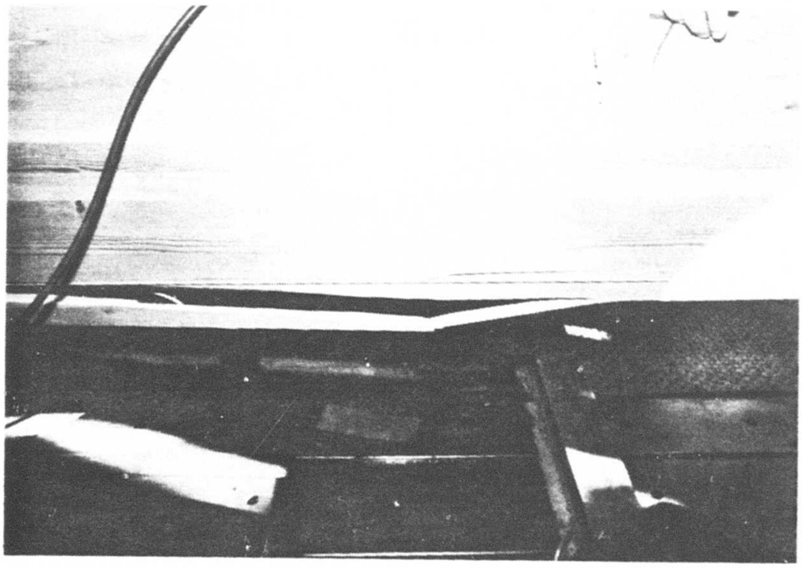


Fig. 3-13. Closeup of Failed Finger Joint and Area of Failure.

Midspan Shore With a 10-inch Long Bearing Plate

Upgrading for this test consisted of shoring each of the two glulams at midspan with a 3/4" x 8" x 10" long bearing plate atop an 8x8 railroad tie. The ends of the glulam beams were supported on beam seats as in the previous base case. The loading arrangement for this test and the deflection gauge locations are shown on Figure 3-14.

Load was stepped slowly to failure in 6,000 lb per ram load increments. Failure occurred when the average applied load reached 65,800 lb per ram, or an equivalent uniform load of 6,823 plf per beam. The total combined dead plus live load at failure was 6,869 plf. The test was performed in 1 hour and 20 minutes, and the duration of load adjustment is 1.45 for this test (see Figure 3-9). Normalizing the load to a 10-year duration reduces the load at failure to 4,736 plf. This represents an increase in load-carrying capacity of 369% over the unshored base case. The load vs midspan deflection curve shown in Figure 3-15 represents the amount of deflection (crushing) that occurred at the midspan shores. The applied loads at failure and the resulting shear and moment diagrams are shown in Figure 3-16. In order to validate these diagrams, an additional bearing test was conducted on a nonfailed segment of the beam under the same shore support conditions, and the results of this test are shown in Figure 3-17. The calculated reaction at the shore was 85,400 lb; the bearing test load at the same amount of crushing was 84,800 lb.

The beam failure was due to a combined bearing and shear failure at the midspan shore on one of the beams. Ordinarily, a bearing failure is not sufficient to fail a beam; however, in this test it contributed to an eventual shear failure when the bearing plate crushed a sufficient thickness of fibers. The other beam did not fail; however, considerable distress due to crushing at the midspan shore was evident. Figure 3-18A shows the test specimen prior to testing. The combined bearing and shear failure at the midspan shore can be seen in Figures 3-18B and 3-19A. The bearing surface on the unfailed beam is shown in Figure 3-19B.

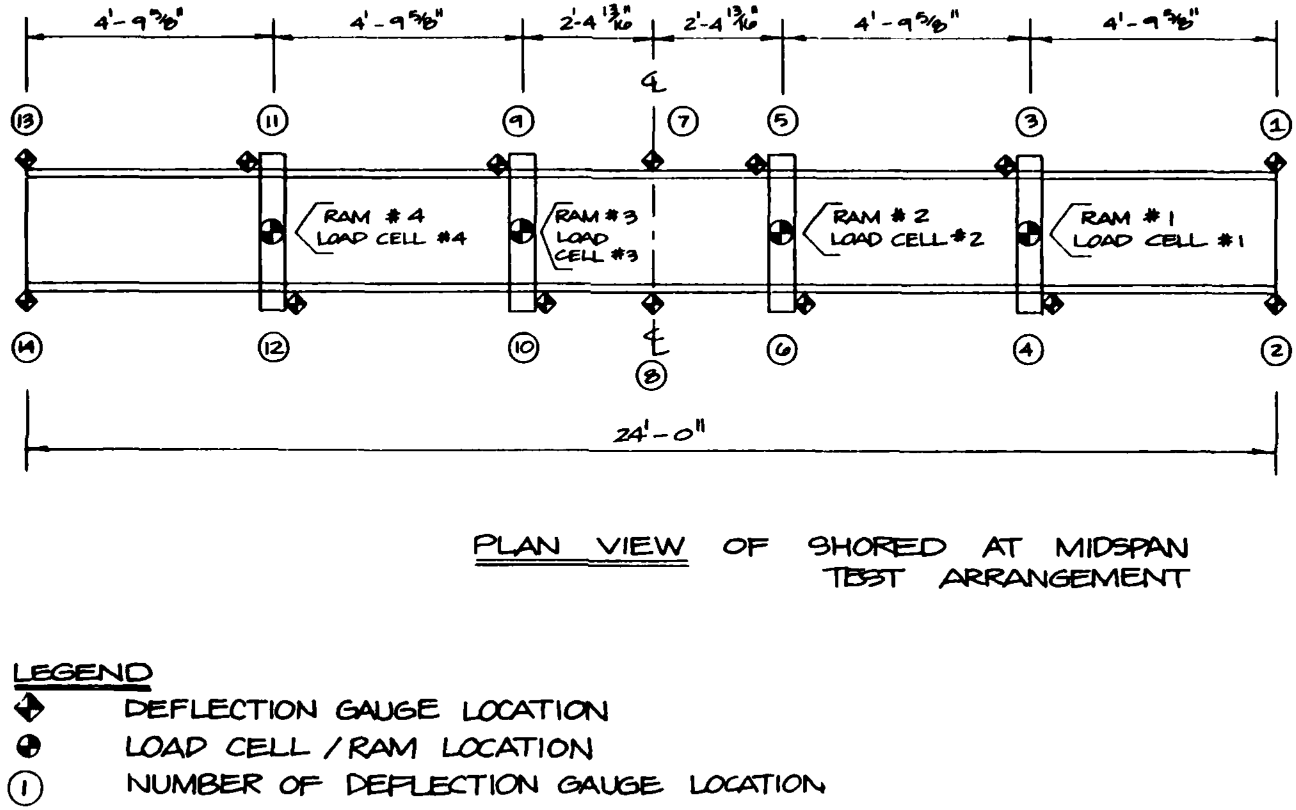


Fig. 3-14. Load Configuration and Deflection Gauge Locations for the Shored at Midspan Glulam Beam Tests.

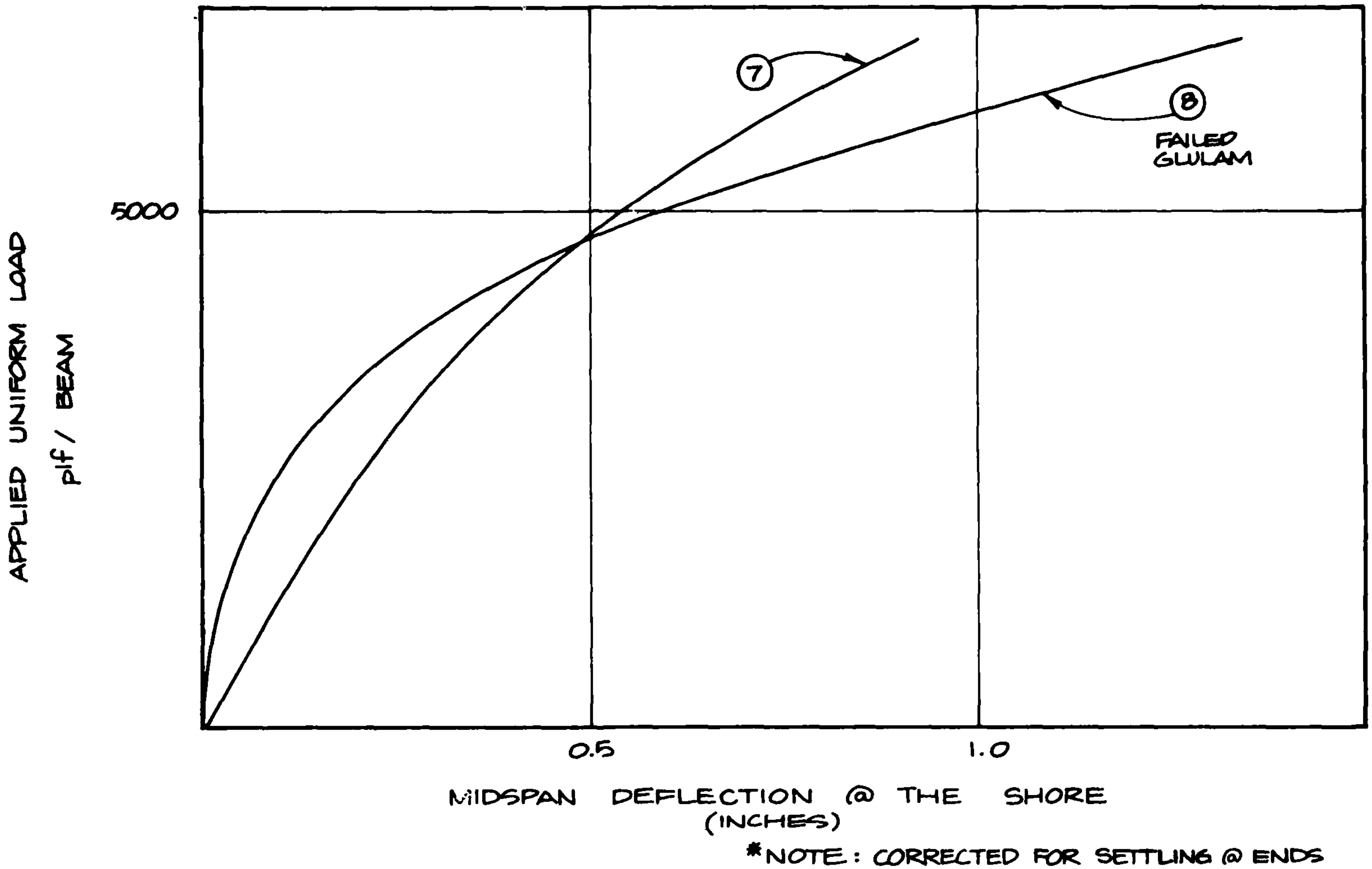


Fig. 3-15. Load vs Midspan Deflection for the Shored at Midspan Test Specimen.

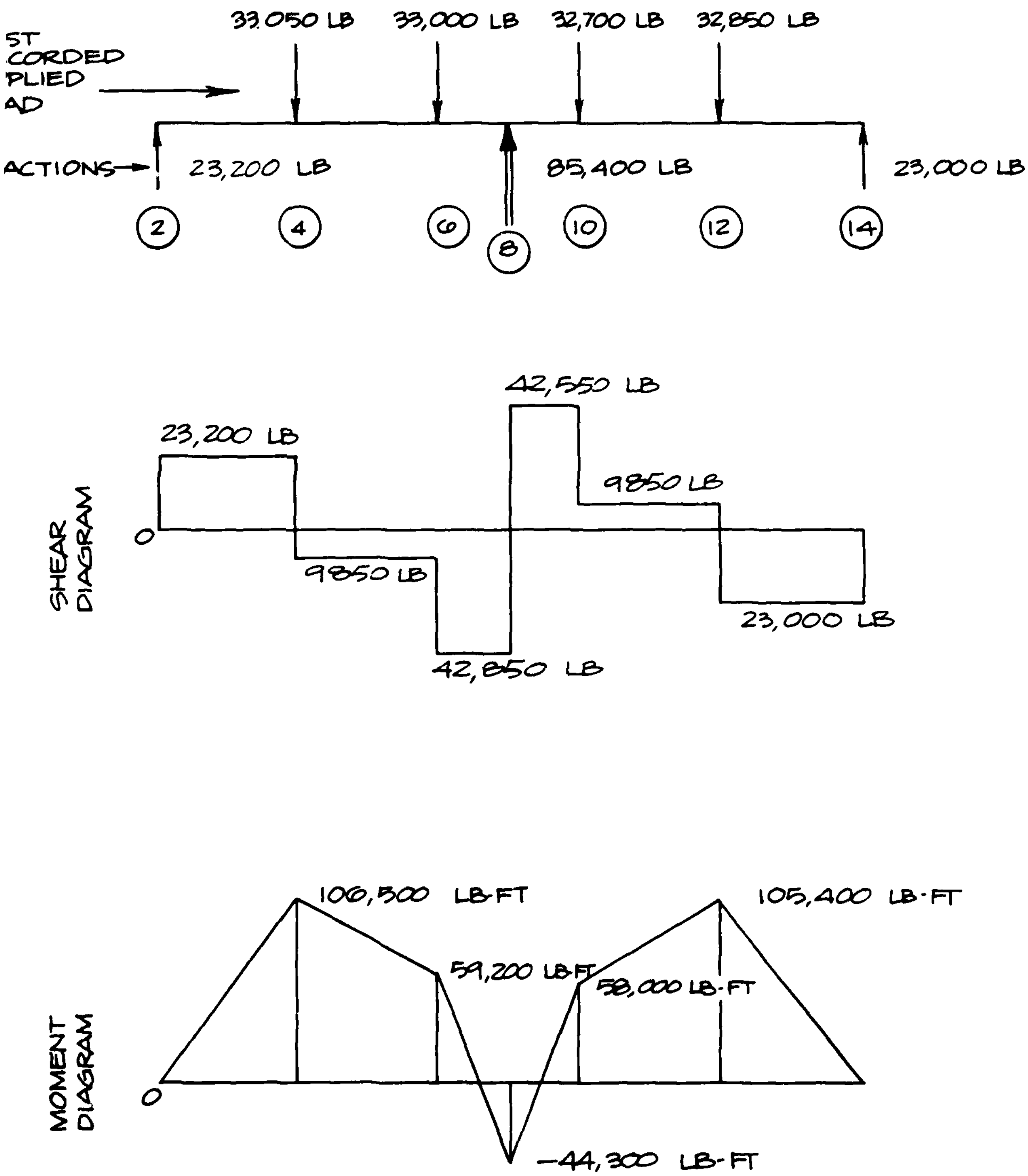


Fig. 3-16. Load, Shear, and Moment Relationships for the Failed Glulam.

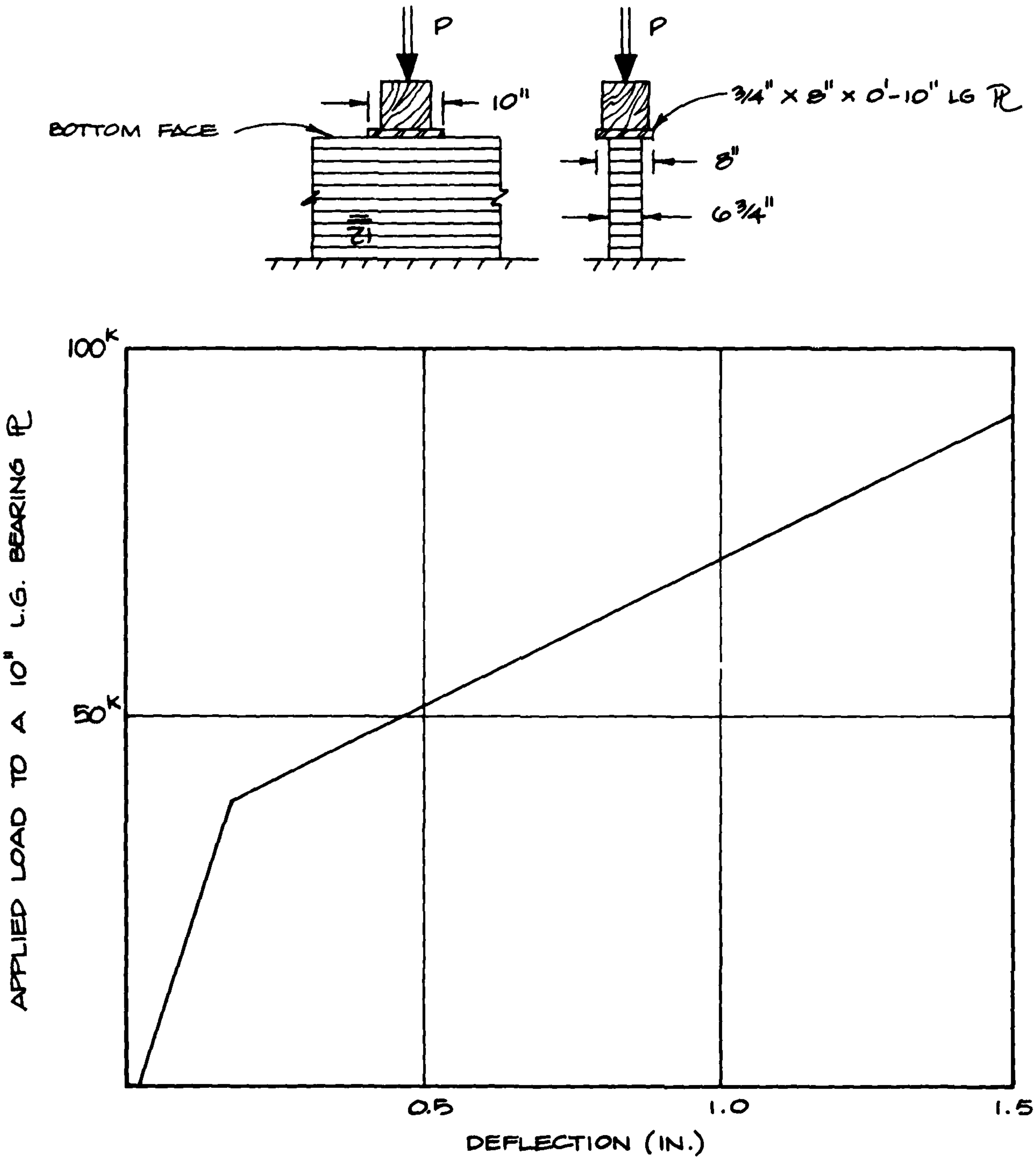
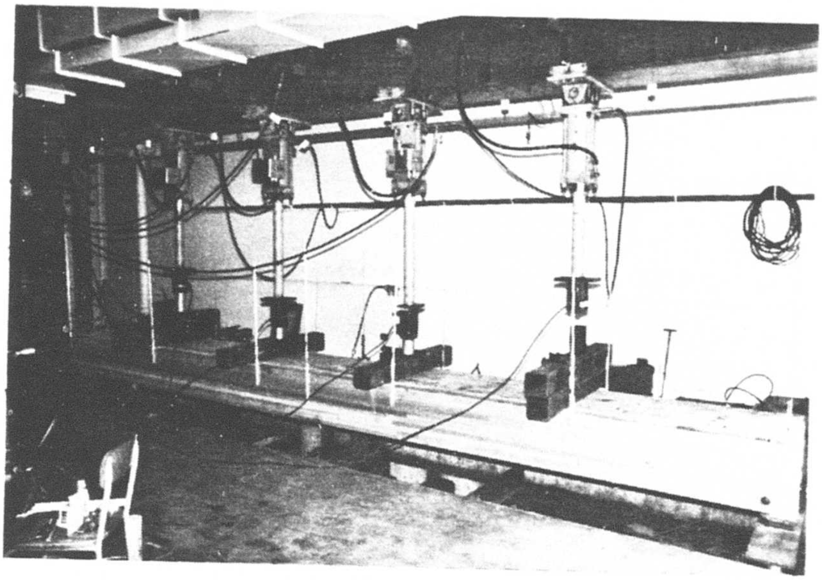


Fig. 3-17. Plot of Bearing Test Results for a Section of the Failed Beam.

A



B

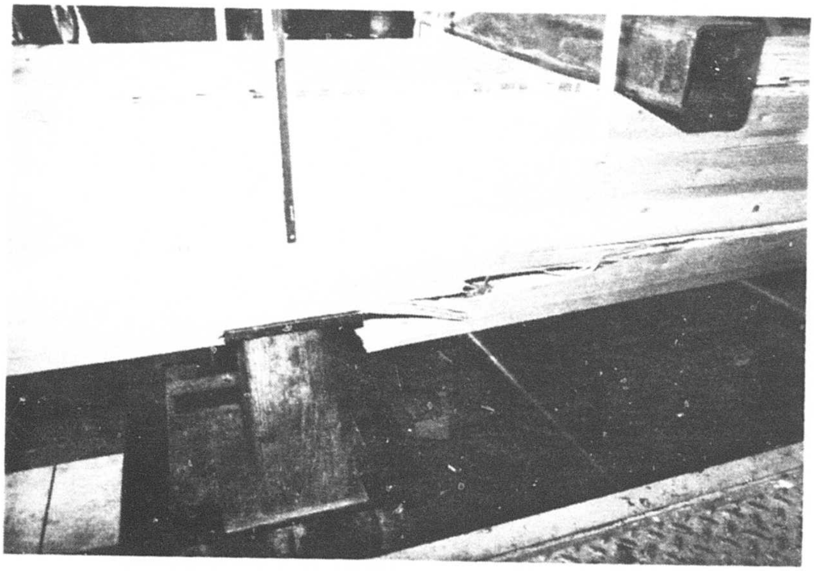
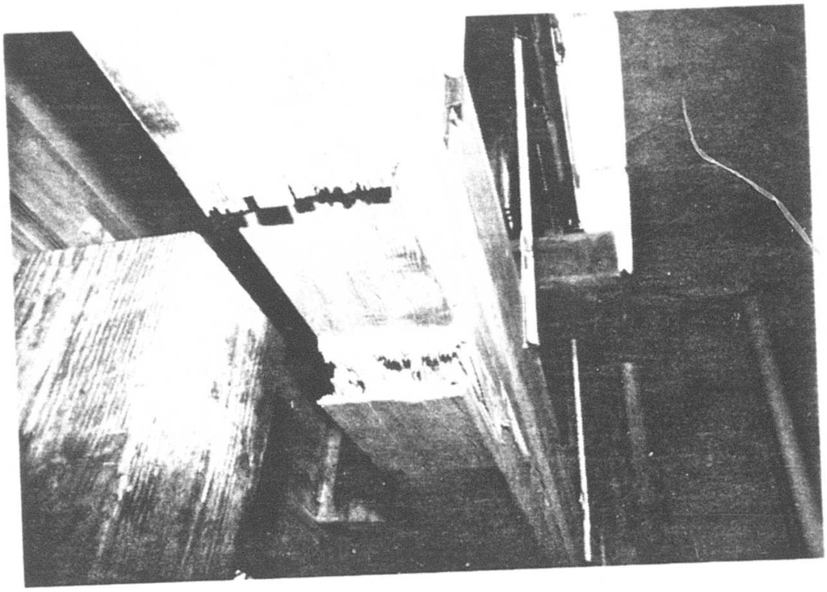
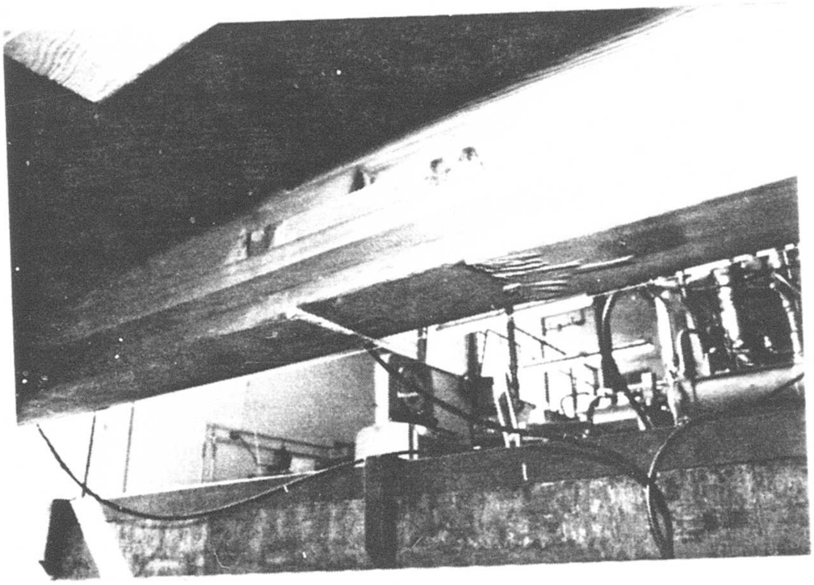


Fig. 3-18. Midspan Shore With 10-Inch Long Bearing Plate, Showing Bearing/Shear Failure.



A



B

Fig. 3-19. Failed and Unfailed Bearing Surfaces at Midspan.

Midspan Shored With a 14-inch Long Bearing Plate

Upgrading for this test was similar to the previous test except that a 14-in. long bearing plate was used. Load cells were placed at shore locations in lieu of the railroad ties in order to determine directly the load carried by each midspan shore. Again, four rams were used to load the beams to failure (see Figure 3-14).

Load was stepped slowly to failure in 12,000 lb per ram increments. Failure occurred when the average applied load reached 64,500 lb per ram, or an equivalent uniform load of 6,248 plf per beam. The combined dead plus live load at failure was 6,292 plf. The test was performed in 1 hour and 37 minutes; the resulting duration of load adjustment is 1.45 for this test (see Figure 3-9). Normalizing to a 10-year duration of load reduces the load at failure to 4,339 plf. This represents an increase in load-carrying capacity of 338% over the unshored base case. The applied uniform load vs reaction in each of the shores at midspan is shown in Figure 3-20. It should be noted that the relationship was linear in both shores. The shear and moment diagrams at failure for the failed and nonfailed beams are shown in Figures 3-21 and 3-22, respectively.

Figure 3-23A shows the test specimen prior to testing. Failure was due to flexure and shear. Early in the test, at about 4,700 plf per beam, an interior tension laminate below deflection gauge 12 failed because of a pre-existing timber break (see Figure 3-23B). Assuming the laminates below the failure were ineffective, the section modulus would be reduced from an original 306 in.^{3} to 203 in.^{3}; in effect, the flexural capacity of the section would be reduced by one-third. Although this localized flexural failure greatly increased beam deflection, the positive moment capacity at this section was sufficient to carry the applied moment for the remainder of the test and was not the direct cause of the glulam's failure. Loading was continued to failure at 6,248 plf per beam, when a shear failure over the midspan shore occurred. Three distinct shear cracks formed over the midspan shore, two of which propagated to the simply supported end at deflection gauge 2's location, see Figure 3-24A. A section was subsequently cut from midspan of the failed beam; photographs of the three shear failure cracks that occurred are shown in Figure 3-24B.

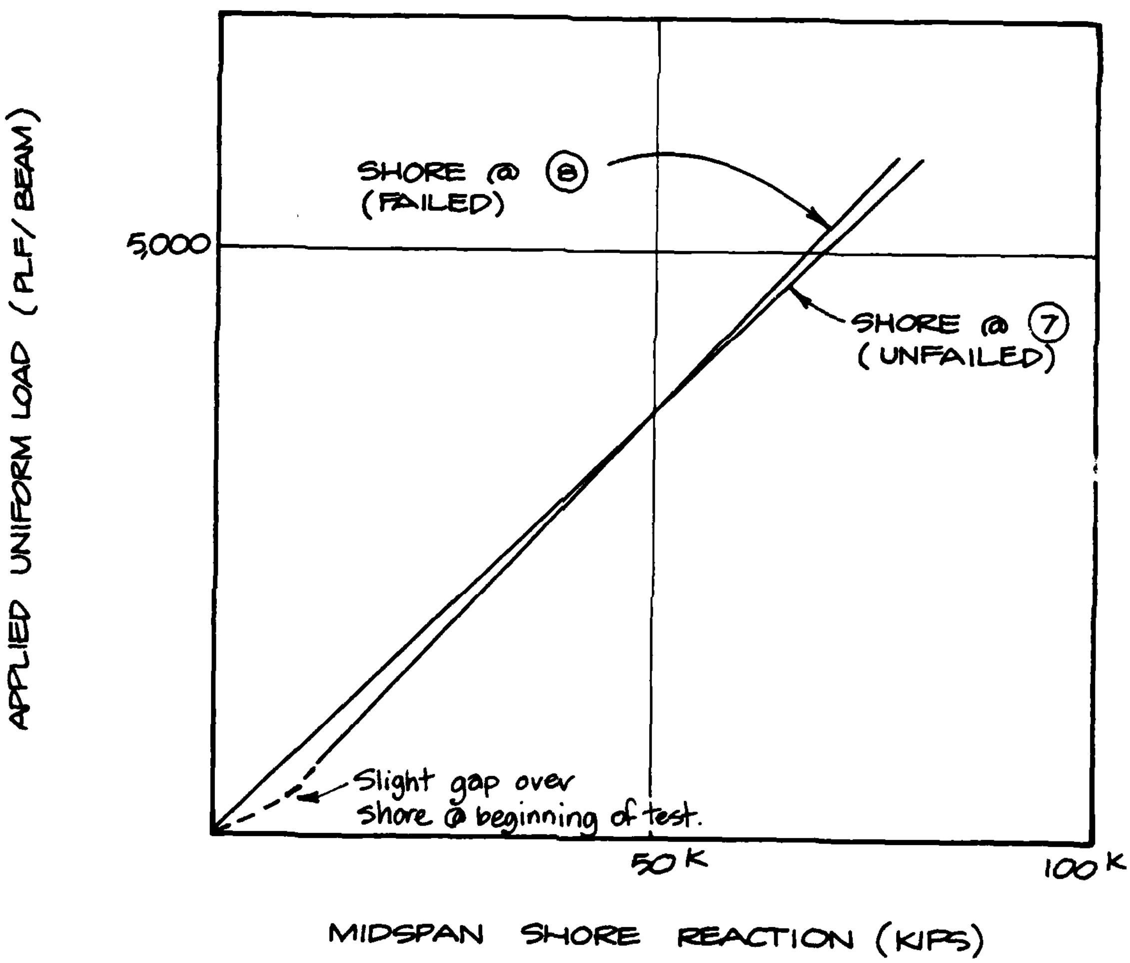


Fig. 3-20. Load vs Midspan Shore Reaction for the Shored at Midspan Test Specimen.

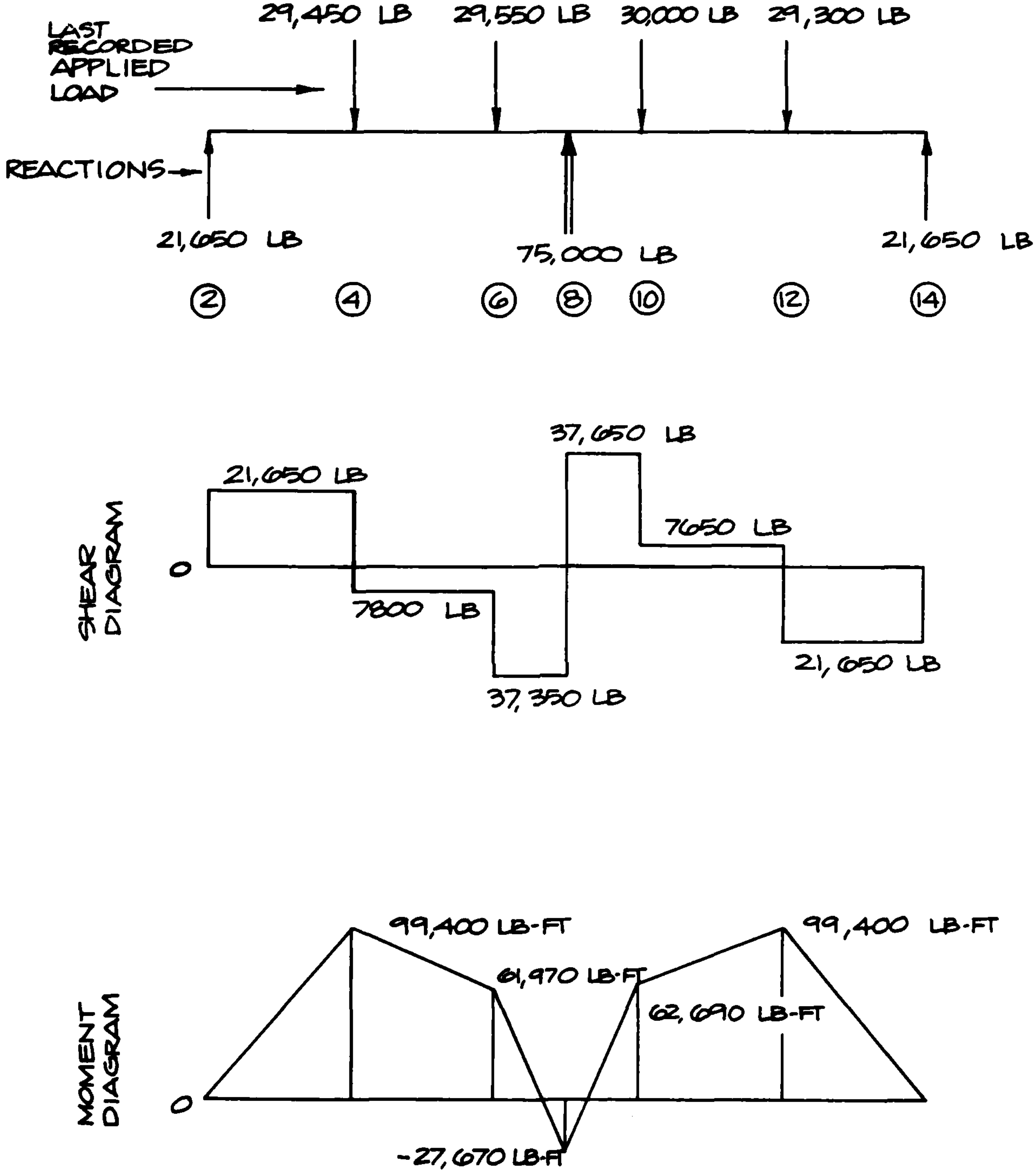


Fig. 3-21. Load, Shear, and Moment Relationships for the Failed Glulam Beam.

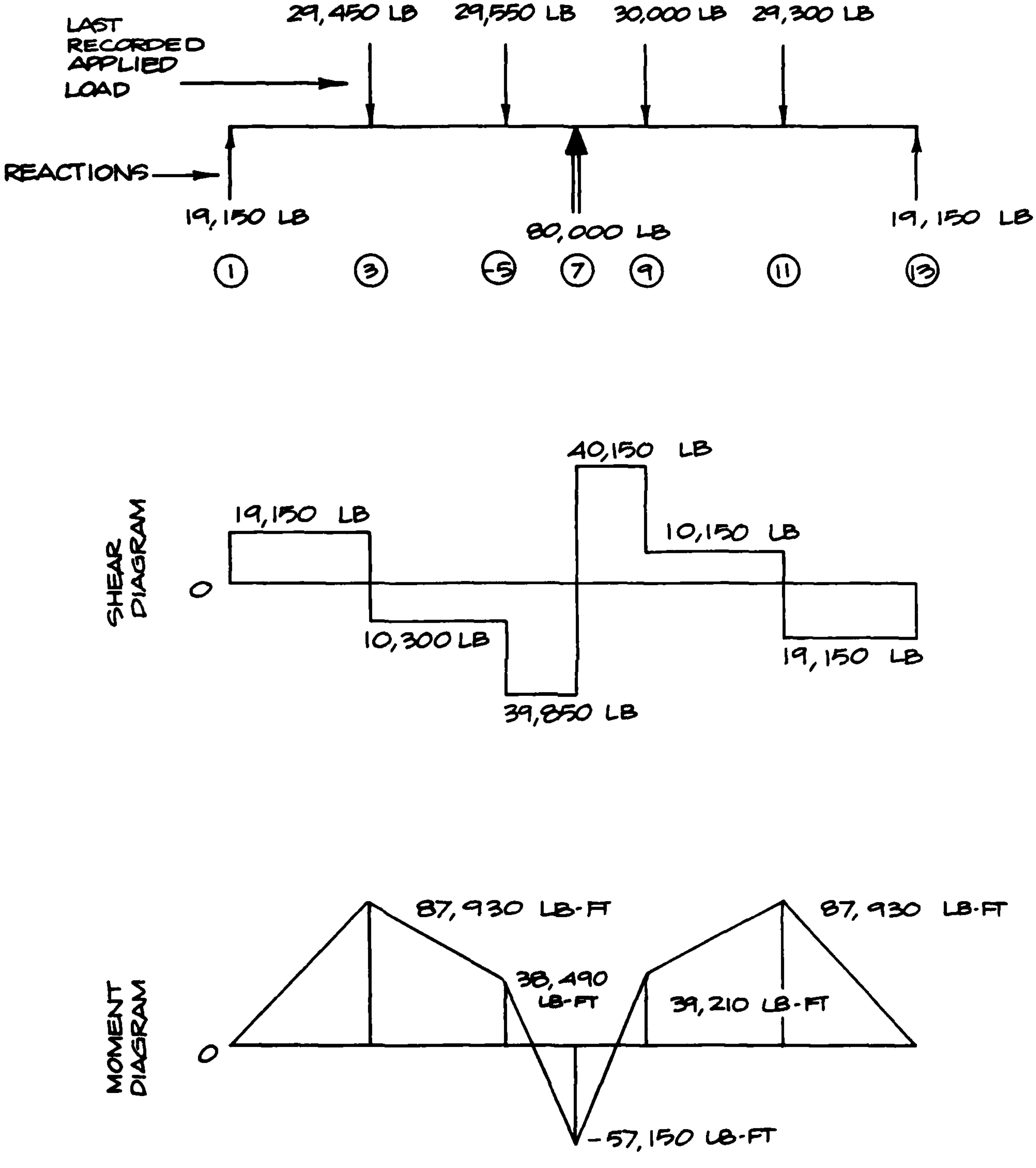
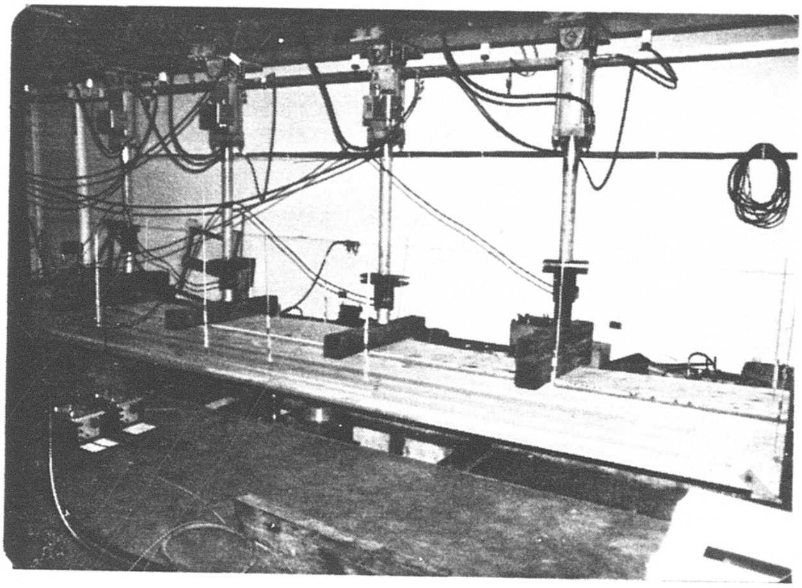


Fig. 3-22. Load, Shear, and Moment Relationships for the Glulam Beam That Did Not Fail.

A



B

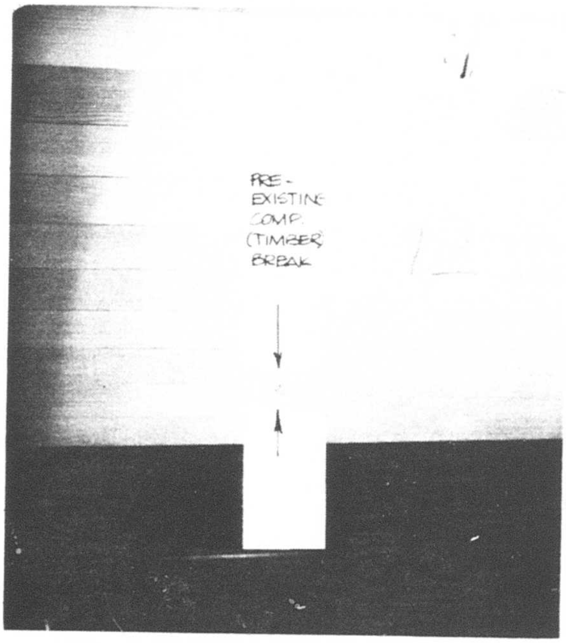
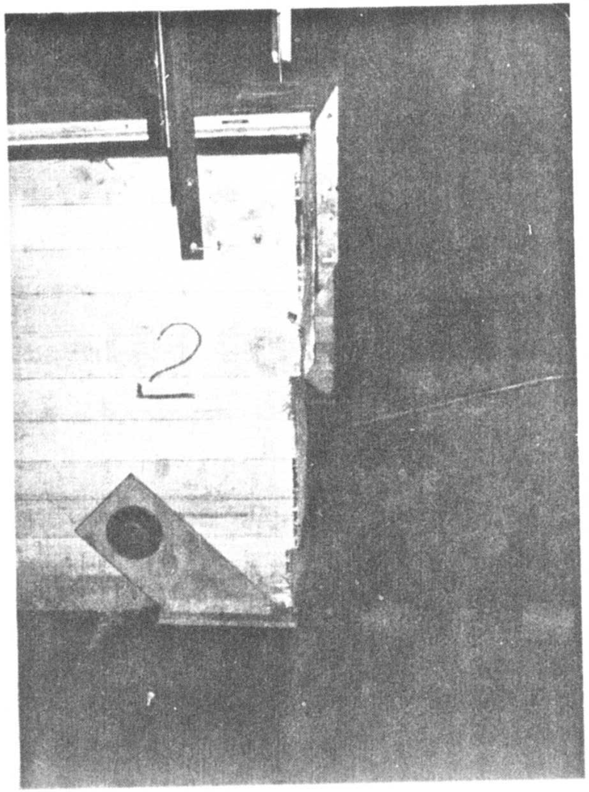
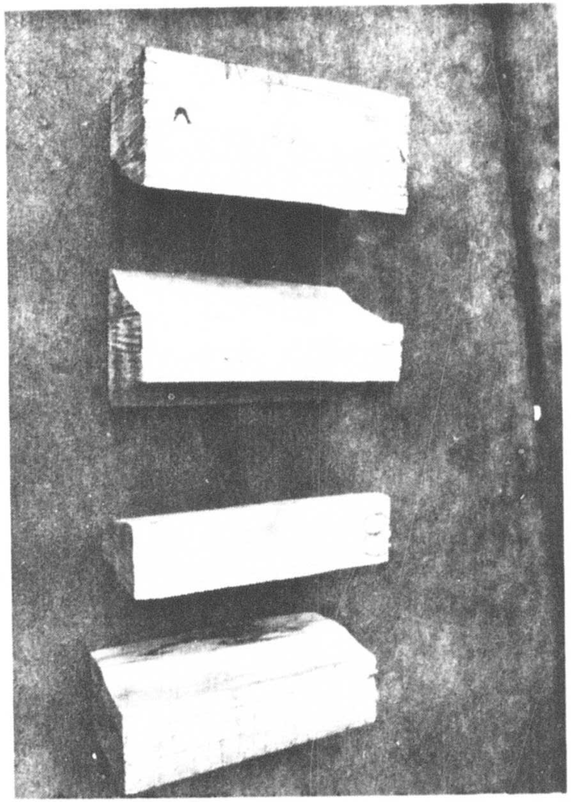


Fig. 3-23. Pretest Photographs of 6-3/4" by 16-1/2" Glulam Beam Test Specimen Shored at Midspan With a 14-Inch Long Bearing Plate. (Note pre-existing timber break).



A. Simple Support After Failure Showing Offsets Due to Shear Failure.



B. Section Over Midspan Shore, Removed After Test, Showing Three Distinct Shear Failures Through Depth of Section.

Fig. 3-24. Posttest Photographs of Failed Beam Shored at Midspan.

TEST SERIES PERFORMED ON 3-1/8" x 18" GLULAM SECTION

Unshored Base Case

This test was on an assembly of two unmodified 3-1/8" x 18" x 24'0" long simply supported glulam beam. The test was performed to determine the load required to fail the test specimen and to establish the basis for comparison of upgrading methods. The ends of the test specimen were supported by four 5" x 10" beam seats. The loading arrangement and deflection gauge locations for this test are shown on Figure 3-8.

Load was stepped slowly to failure in 2,000 lb per ram load increments. Failure occurred when the average applied load was 20,000 lb per ram, or an equivalent uniform load of 1,121 plf per beam. The combined dead plus live load for this beam is 1,145 plf per beam. The test was performed in 61 minutes; the resulting duration of load adjustment factor is 1.48 (see Figure 3-9). Normalizing to a 10-year duration of load reduces the load at failure to 774 plf per beam. The design load for this beam is 471 plf, based upon a 10-year duration of load; thus, the factor of safety against failure for this beam is 1.64. The calculated uniform load vs midspan deflection curve is shown in Figure 3-25. Shear and moment diagrams for the beams at failure are shown in Figure 3-26.

Failure was caused by flexure and occurred approximately 4 feet from midspan, in the peak positive moment zone, of one beam. Failure was the result of tension separation in a finger joint in the bottom laminate of the glulam beam. The other beam showed no evidence of permanent damage and returned to its original position when the load was removed. Figure 3-27A shows the test specimen just prior to testing. Figure 3-27B shows the failed finger joint near deflection gauge number 4. Figure 3-28 shows the center portion of the failed beam and the extensive fracturing that occurred because of the failure of the finger joint.

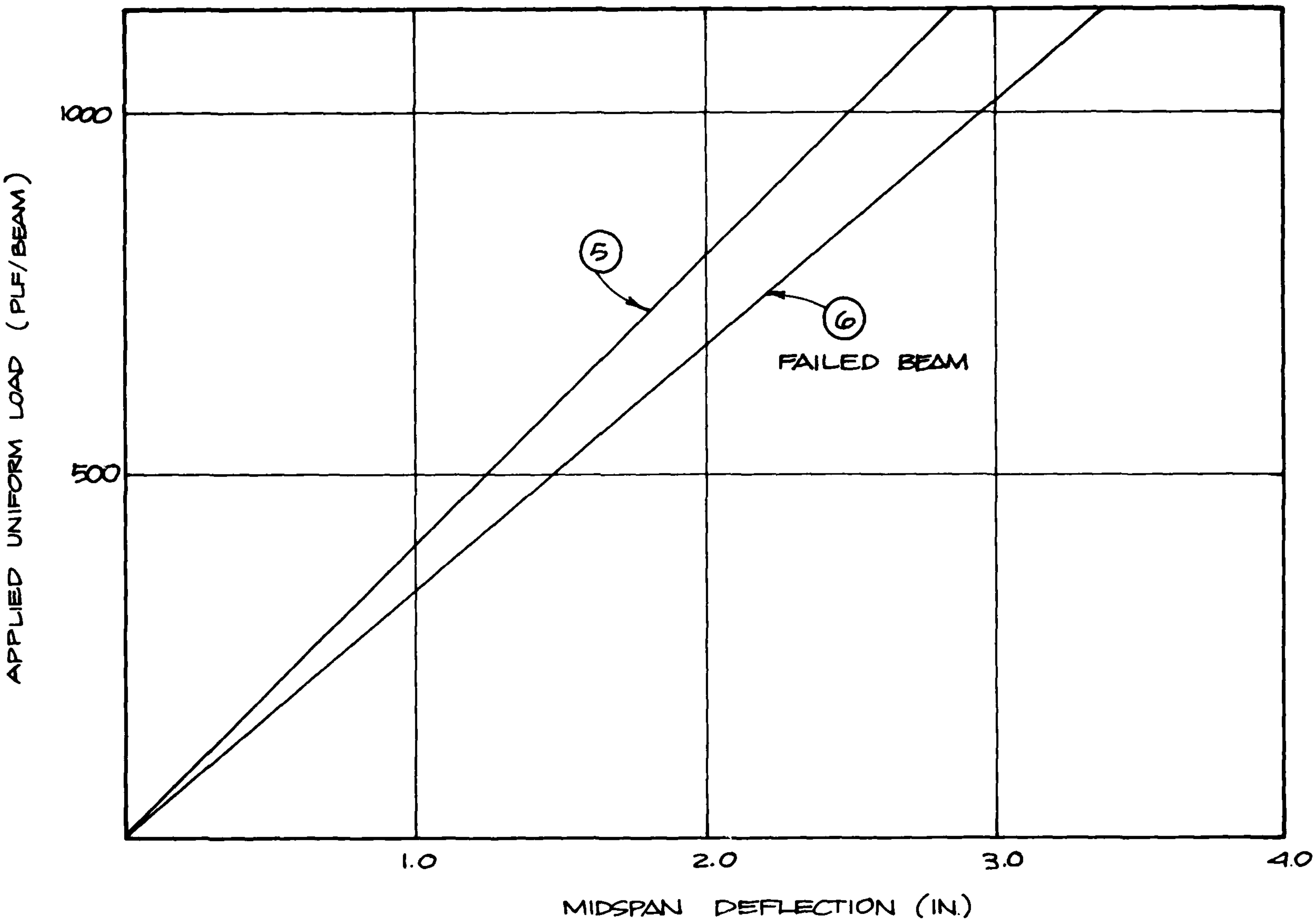


Fig. 3-25. Load vs Midspan Deflection for the 3-1/8" by 18" Glulam Test Specimen.

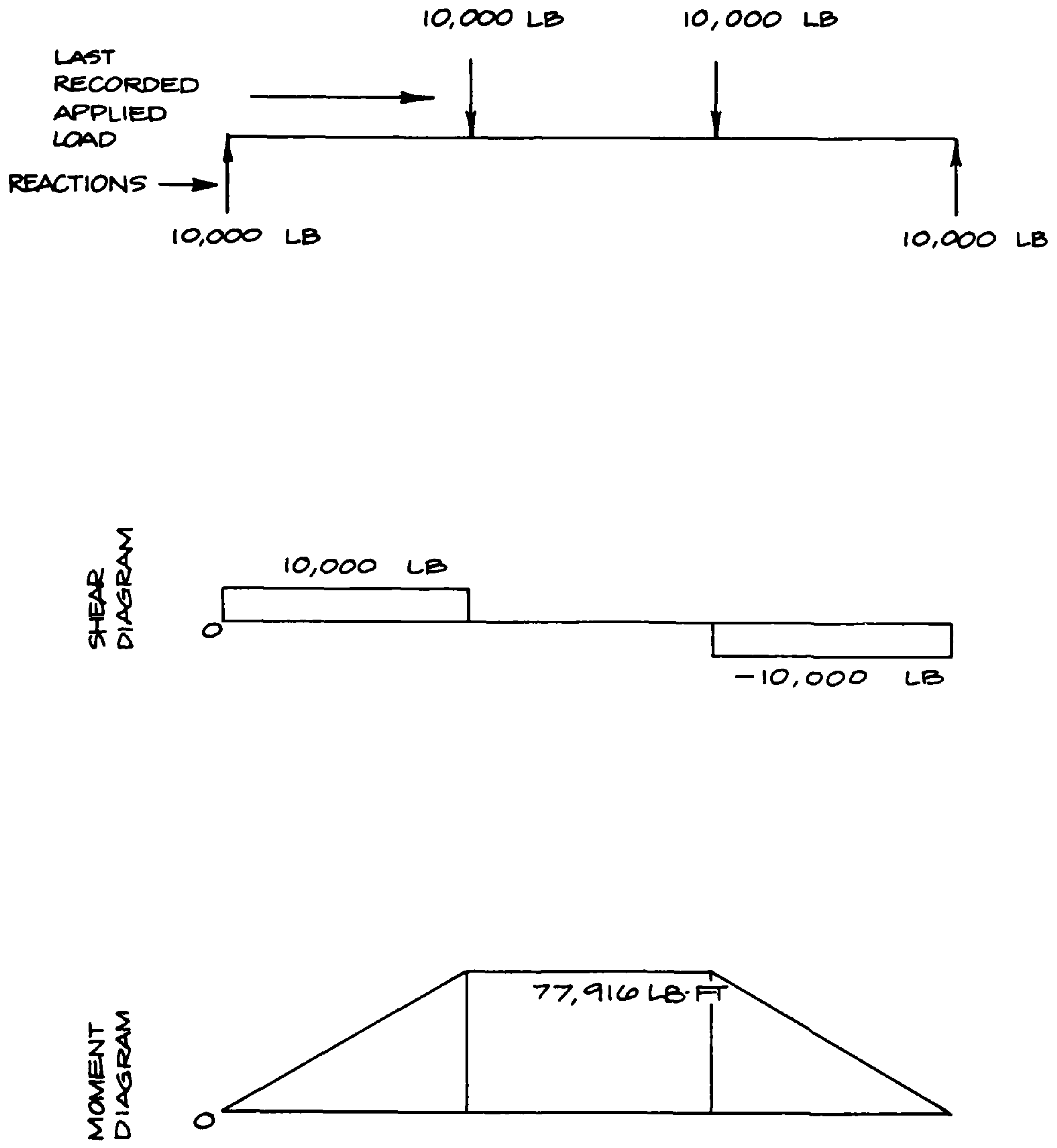
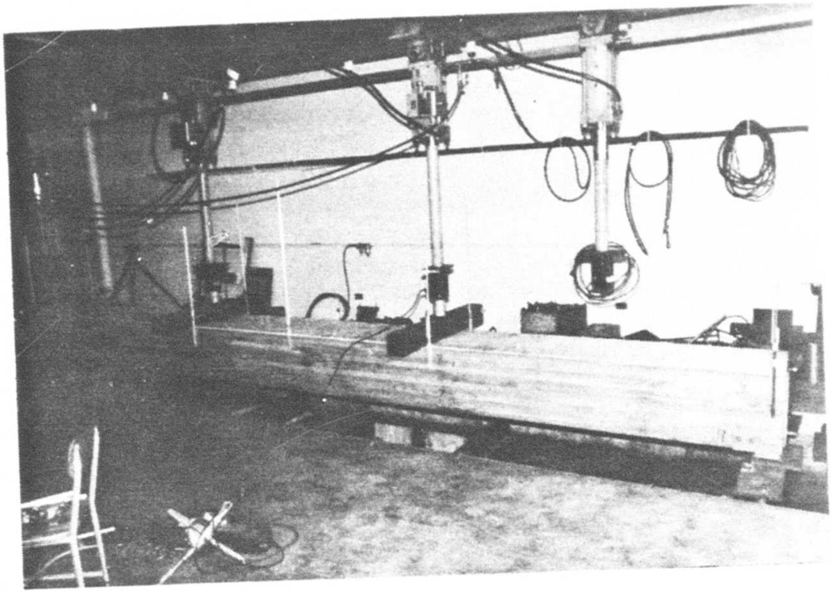
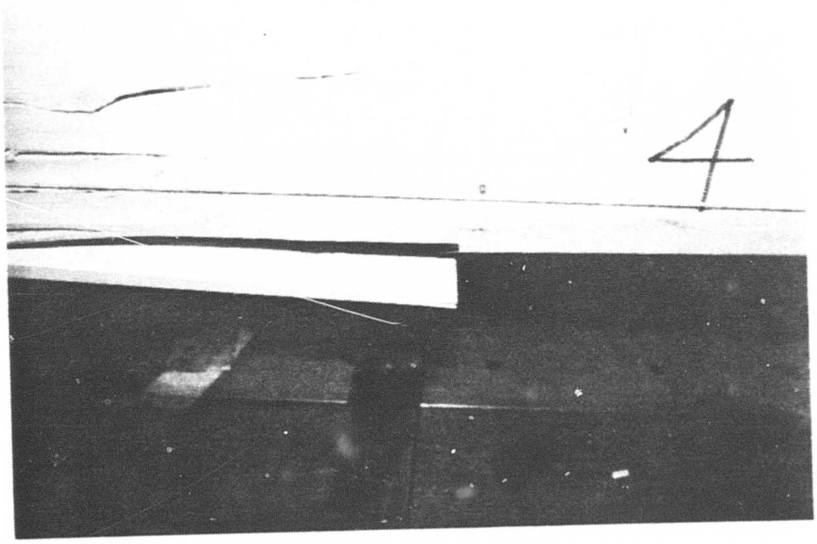


Fig. 3-26. Load, Shear, and Moment Relationships for the 3-1/8" by 18" Glulam Test Specimens.

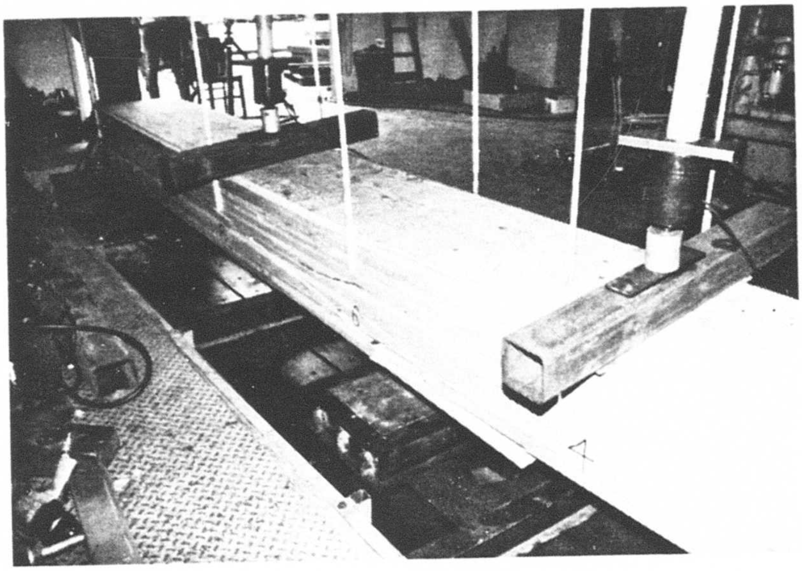


A



B

Fig. 3-27. Pre- and Post-Test Photographs of the 3-1/8" by 18" Base Case Glulam Test Specimen. (Note failed finger joint on the tension laminate of the failed glulam beam)



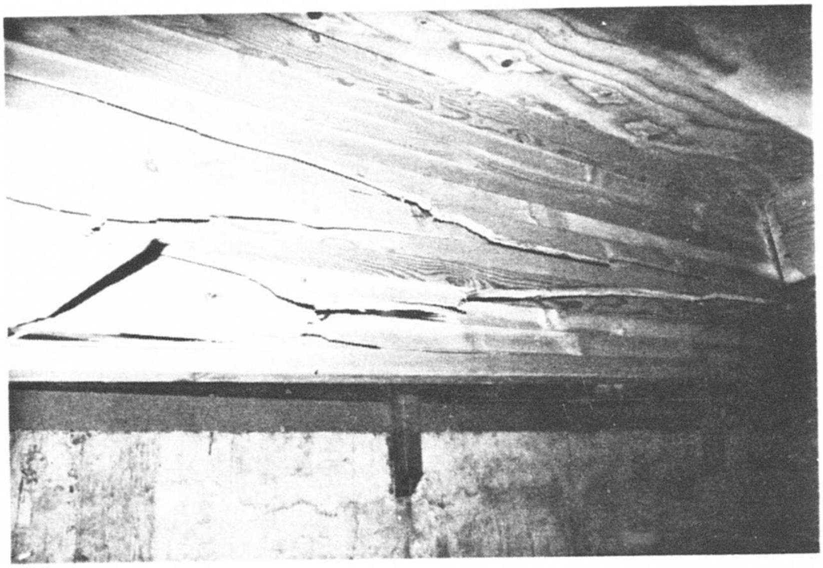


Fig. 3-28. Posttest Photographs of Test Specimen Showing the Failed Glulam Beam at Midspan.

Midspan Shore With a 10-inch Long Bearing Plate

Upgrading for this test consisted of shoring each of the glulams at midspan with a 3/4" x 8" x 0'10" long bearing plate atop a load cell. Four hydraulic rams located at the one-fifth points were used to load the test beams (see Figure 3-14). The ends of the glulam beams were supported on standard beam seats as in the previously described base case test.

Load was stepped slowly to failure in 4,000 lb per ram load increments. Failure occurred when the average applied load reached 34,700 lb per ram, or an equivalent uniform load of 3,308 plf per beam. The total combined dead plus live load at failure was 3,337 plf. The test was performed in 1 hour and 25 minutes; the resulting duration of load adjustment is 1.45 for this test. Normalizing to a 10-year duration reduces the load at failure to 2,301 plf. This represents an increase in load-carrying capacity of 291% over the unshored base case. The uniform load vs midspan deflection curve shown in Figure 5-29 represents the amount of deflection (crushing) that occurred at the midspan shores. The load versus midspan shore reaction was plotted and is shown in Figure 3-30. The applied load at failure and the resulting shear and moment diagrams are shown in Figures 3-31 and 3-32.

Failure was due to a combination of bearing and flexure. At about 16,000 lb per ram, a localized bearing failure occurred over the midspan shore of one beam. As the load was increased, the bearing plate continued to crush the lower flange of the glulam until a horizontal crack in the bottom laminate of the beam propagated into the positive moment zone, and a flexural failure occurred. The crack extended from deflection gauge 7 at midspan to just beyond deflection gauge 11's location (see Figure 3-32). Figure 3-33 shows the test section prior to the test. Closeups of the bearing-flexural failure are shown in Figure 3-34 and posttest views of both crushed bearing surfaces at midspan in Figure 3-35.

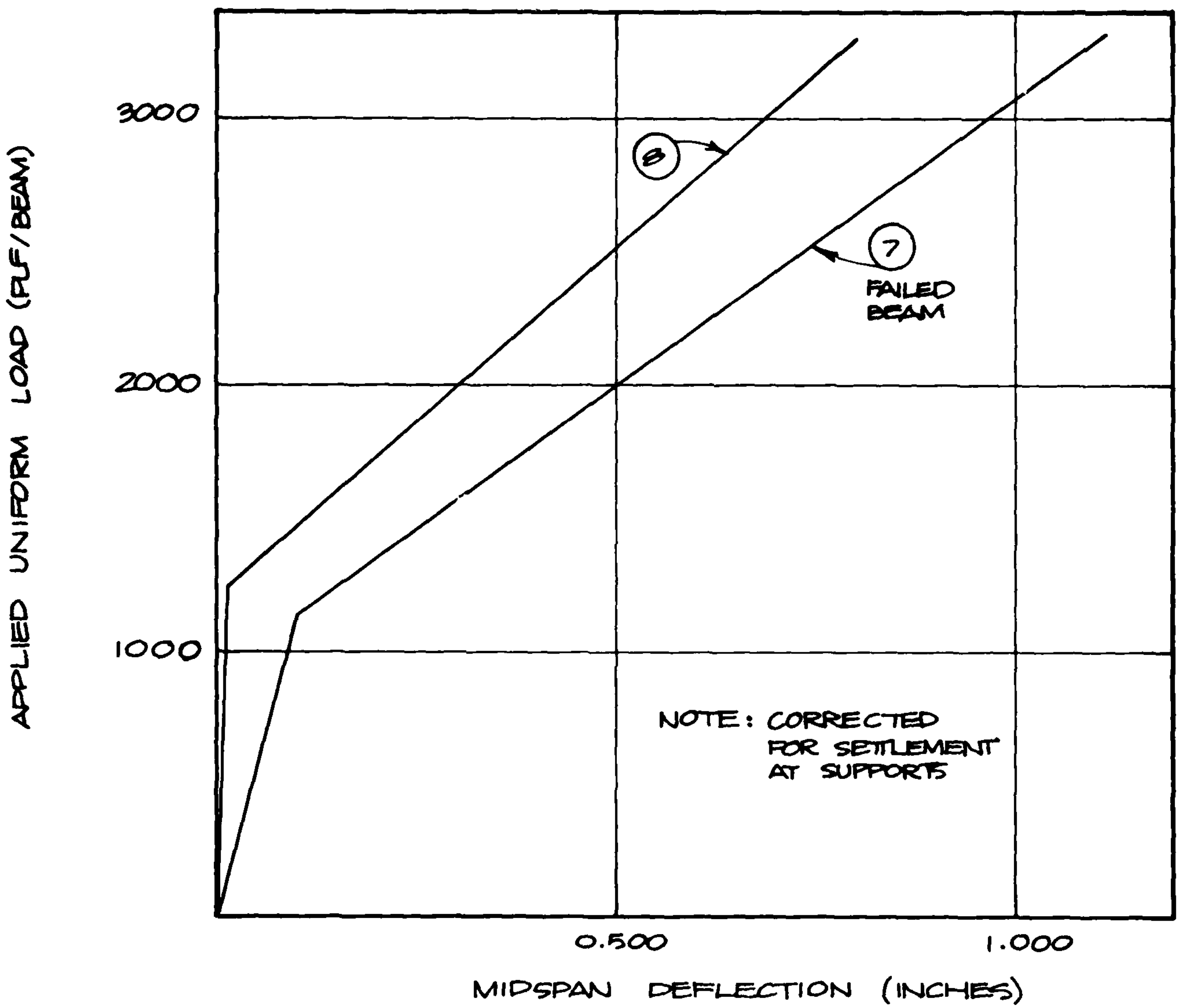


Fig. 3-29. Load vs Midspan Deflection of the 3-1/8" by 18" Glulam Test Specimen Shored at Midspan With a 10-Inch Long Bearing Plate.

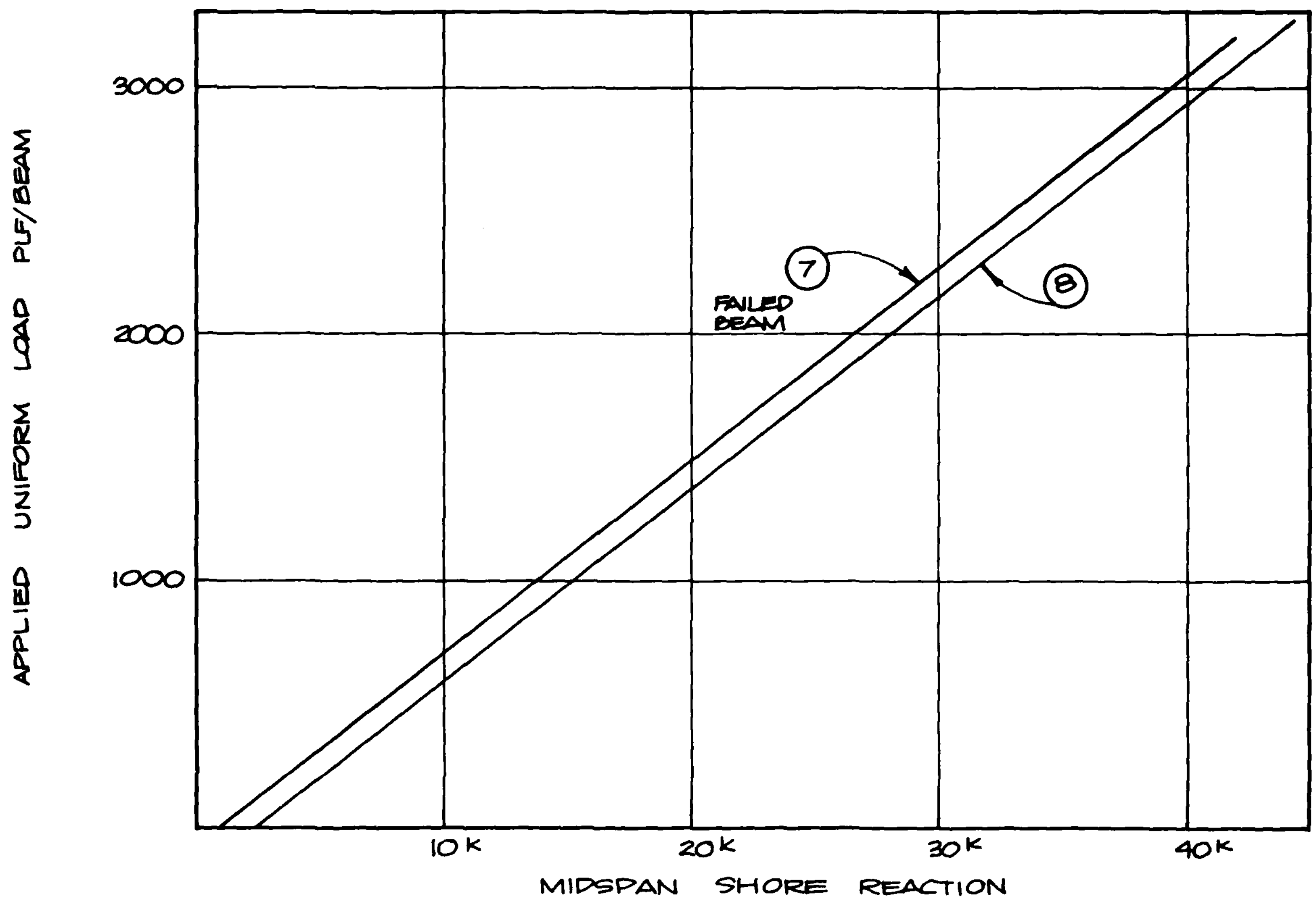


Fig. 3-30. Load vs Midspan Shore Reaction for the Shored Glulam Test Specimen.

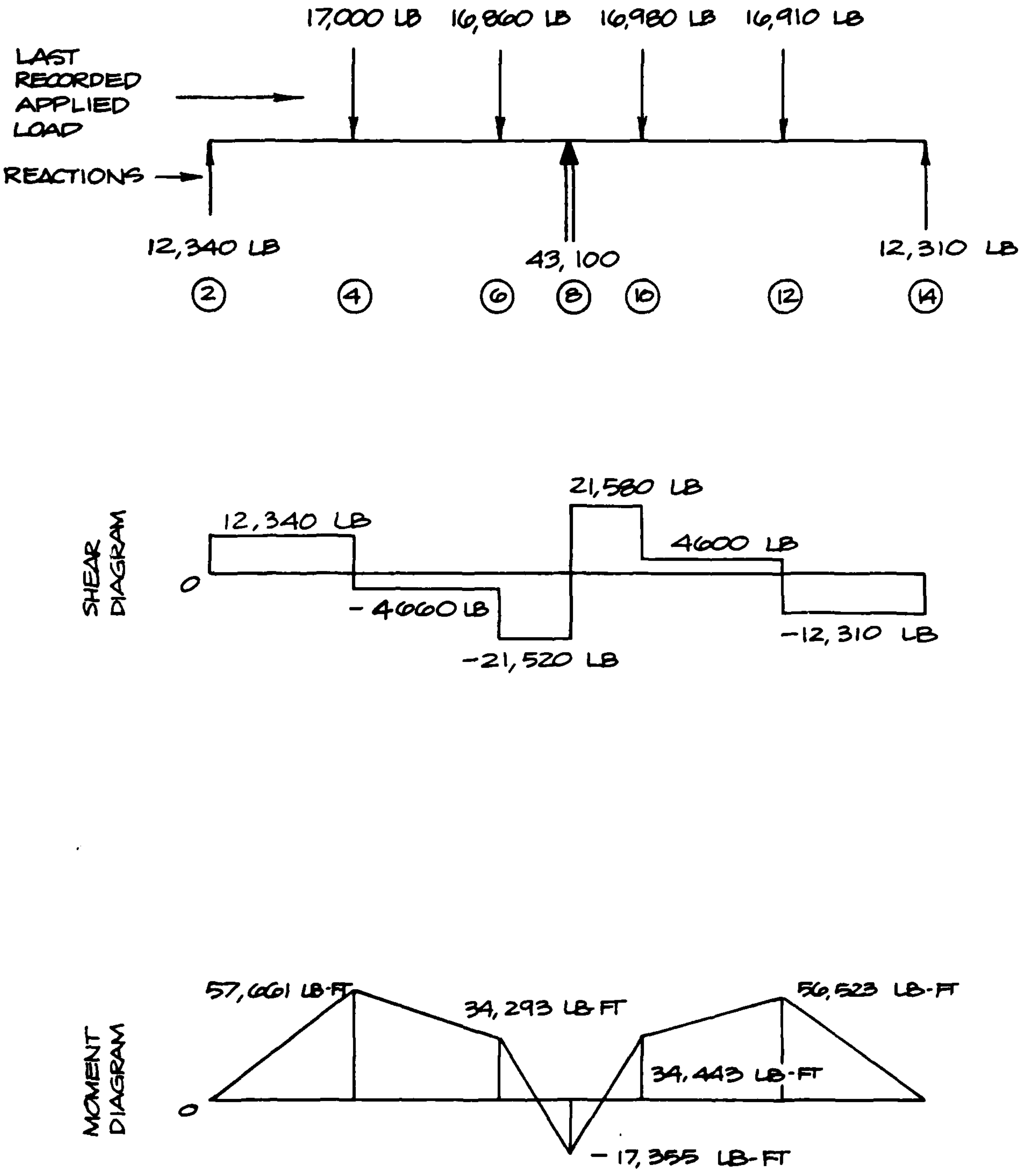


Fig. 3-31. Load, Shear, and Moment Relationships for the 3-1/8" by 18" Glulam Beam Test Specimen, Shored at Midspan With 10-Inch Long Bearing Plate for the Glulam Beam That Did Not Fail.

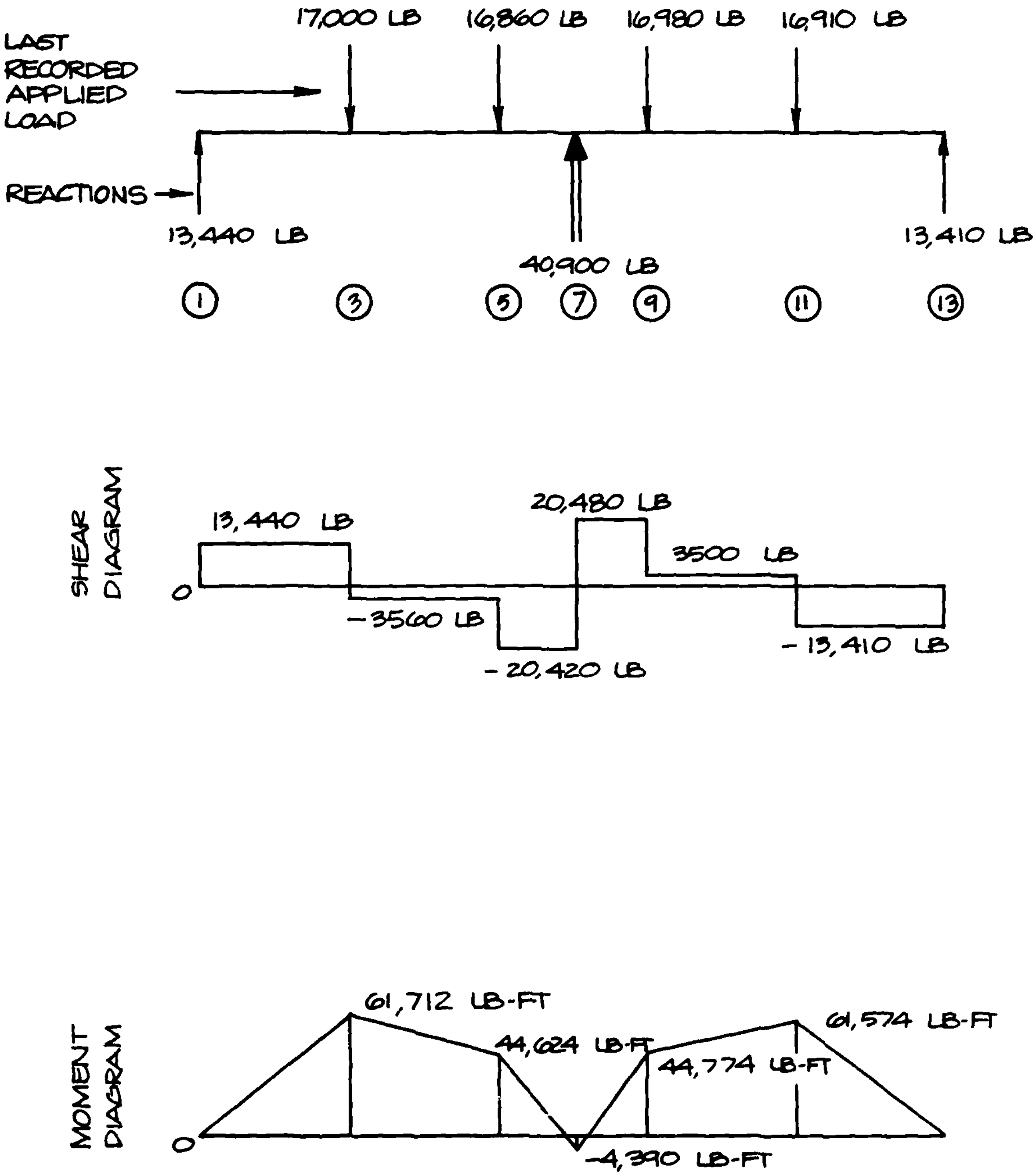


Fig. 3-32. Load, Shear, and Moment Relationships for the 3-1/18" by 18" Glulam Beam Test Specimen Shored at Midspan With 10-Inch Long Bearing Plate, for the Failed Glulam Beam.

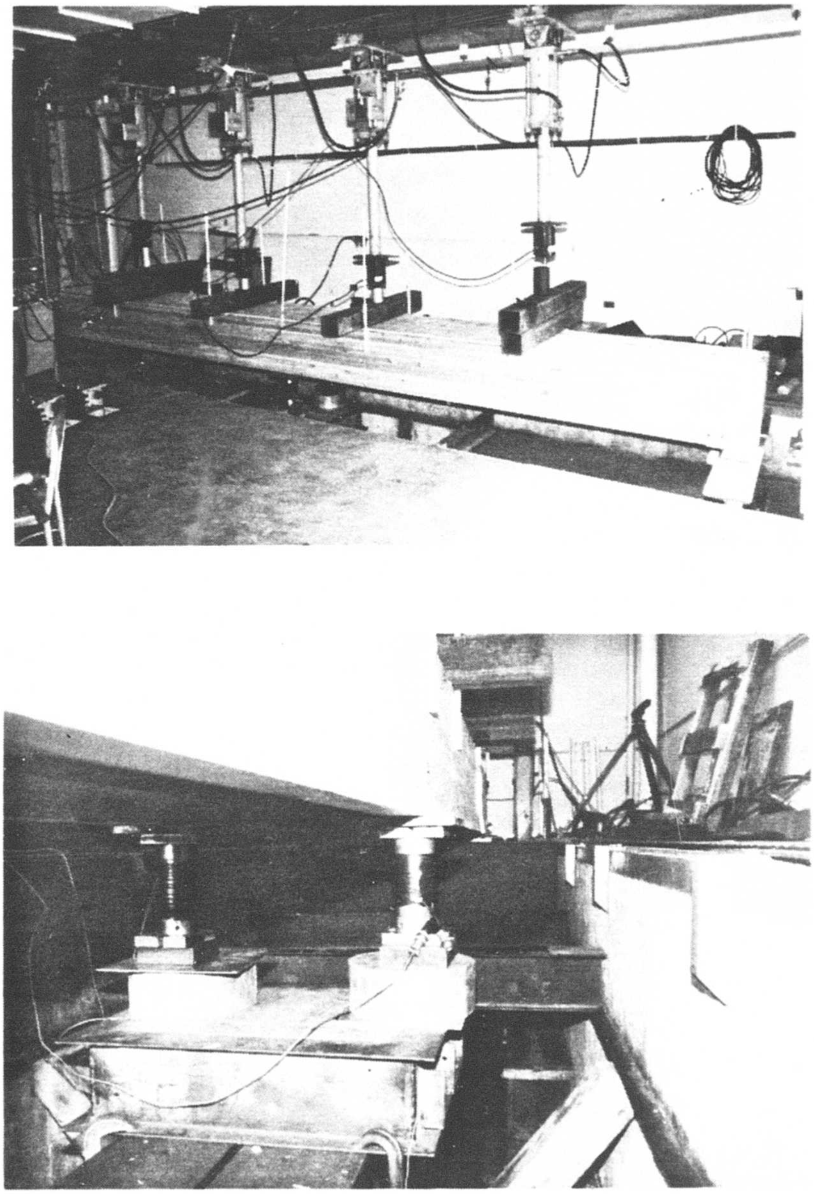


Fig. 3-33. Pretest Photographs of Test Section.

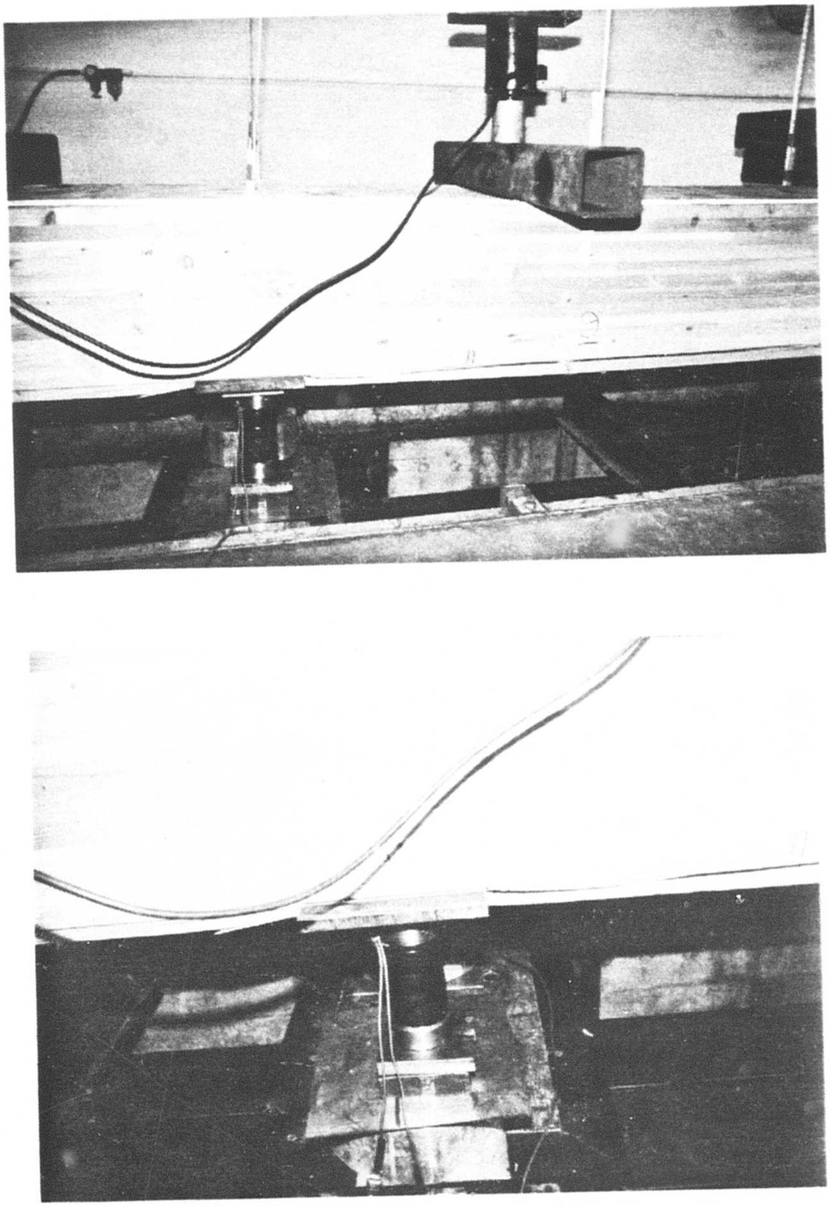
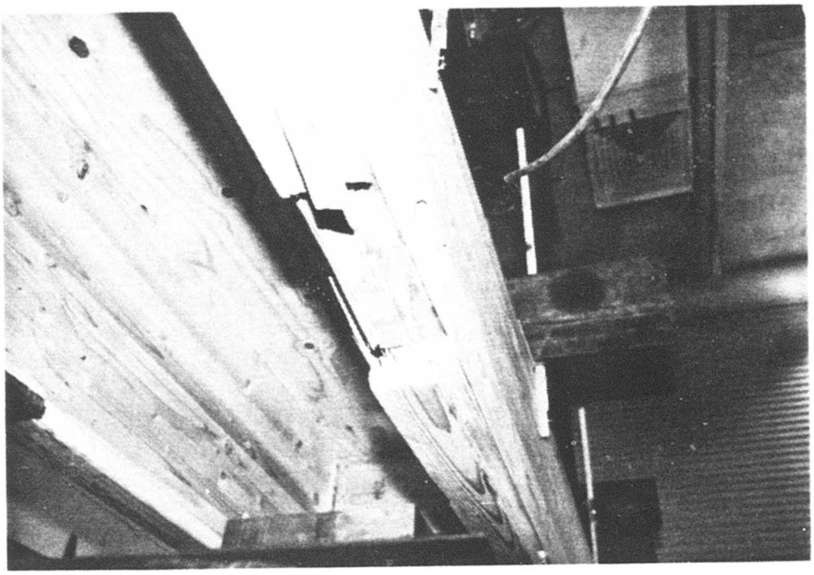


Fig. 3-34. Closeups of Bearing-Flexural Failure at the Midspan Shore on the Failed Glulam Beam.



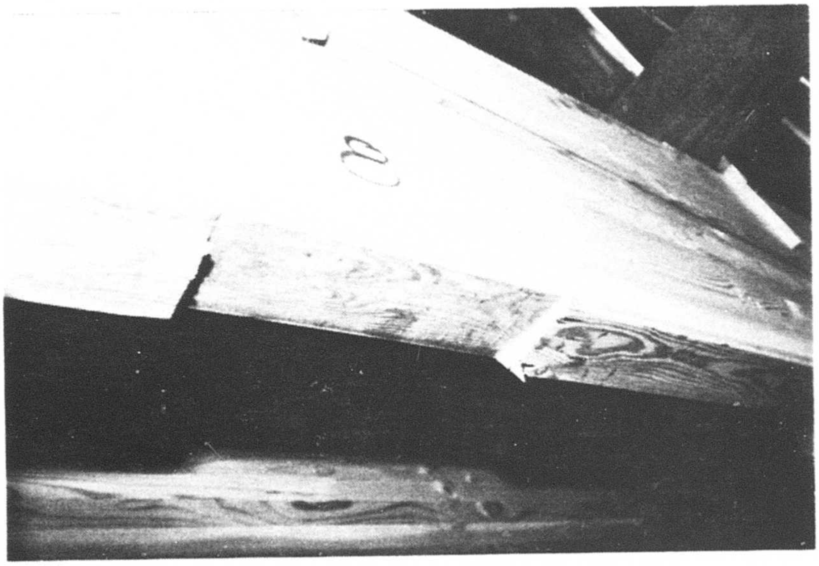


Fig. 3-35. Posttest Photographs of Crushed Bearing Surfaces at Midspan.

Midspan Shore With a 14-inch Long Bearing Plate

Upgrading for this test was similar to the previous test, except that a 14-in. long bearing plate was used. Load cells supported the bearing plates in the same manner as the previous test. Four hydraulic rams, located at the one-fifth points were used to load the beams (see Figure 3-14).

Load was stepped slowly to failure in 4,000 lb per ram load increments. Failure occurred when the average load reached 33,500 lb per ram, or an equivalent uniform load of 3,234 plf per beam. The combined dead plus live load at failure was 3,263 plf per beam. The test was performed in 57 minutes; the resulting duration of load factor is 1.48 for this glulam beam test. Normalizing to a 10-year duration reduces the load at failure to 2,204 plf per beam. This represents an increase in load-carrying capacity of 285% over the unshored base case. The applied uniform load vs midspan deflection for both beams tested is shown in Figure 3-36. Additionally, the equivalent applied uniform load vs shore reactions have been plotted and are presented in Figure 3-37. The shear and moment diagrams at failure for both beams are shown in Figures 3-38 and 3-39.

Failure was due to a combination of bearing, shear, and flexure. A localized bearing failure over the shore on one beam led to a shear crack propagating toward the support at deflection gauge 2. At about the same time, the lower two laminates at deflection gauge 12 failed in tension at the finger joints. The positive moment occurring at deflection gauge 12's location produced the flexural failure in the tensile laminates at this section of the beam. Figure 3-40A shows the entire test section prior to testing. Figures 3-40B and 3-41A show the bearing-shear failure over the midspan shore, and Figures 3-41B and 3-42 show the flexural-finger joint failure in the vicinity of deflection gauge 12.

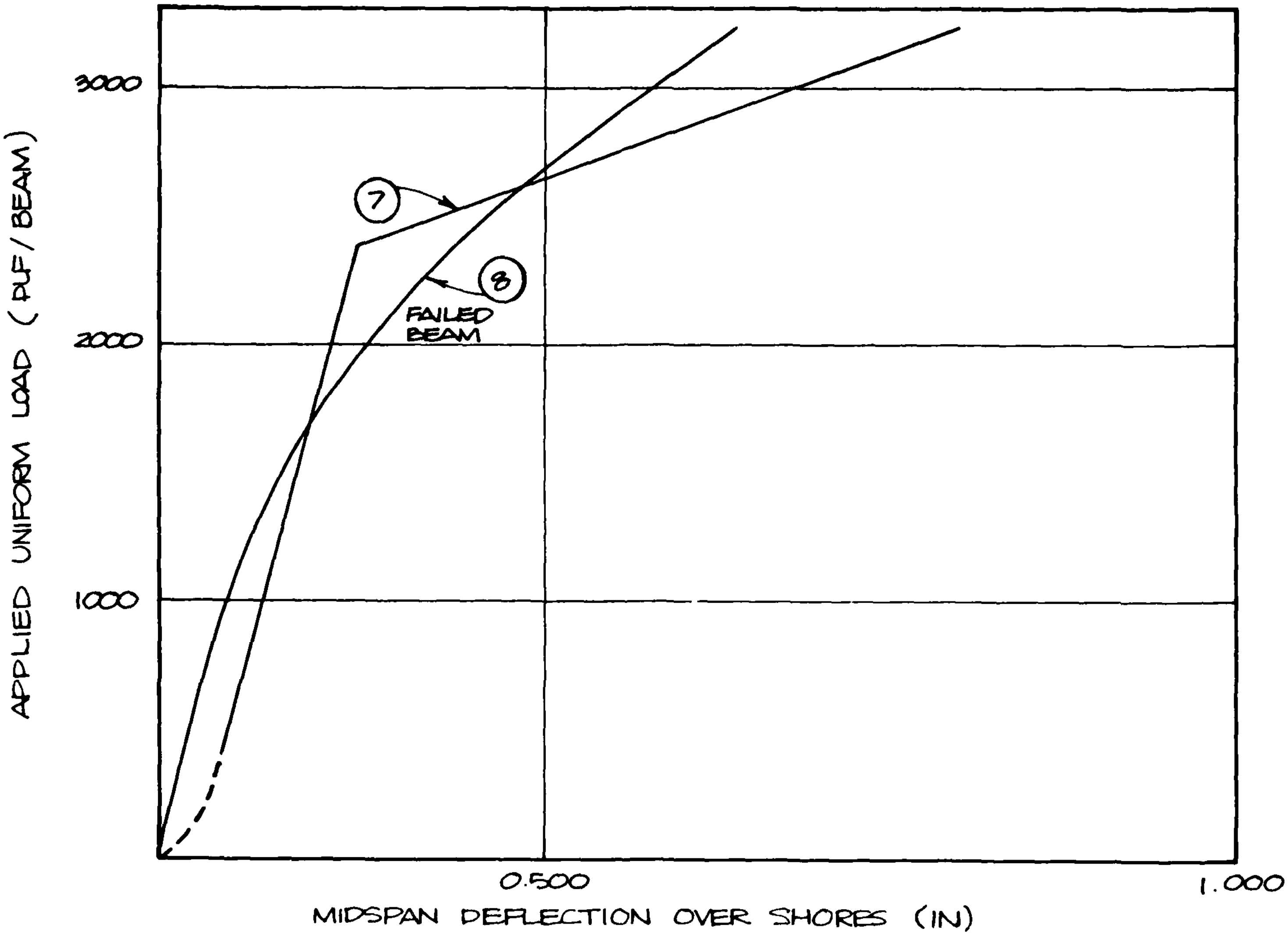


Fig. 3-36. Load vs Midspan Deflection for the 3-1/8" by 18" Glulam Test Specimen Shored at Midspan With a 14-Inch Long Bearing Plate.

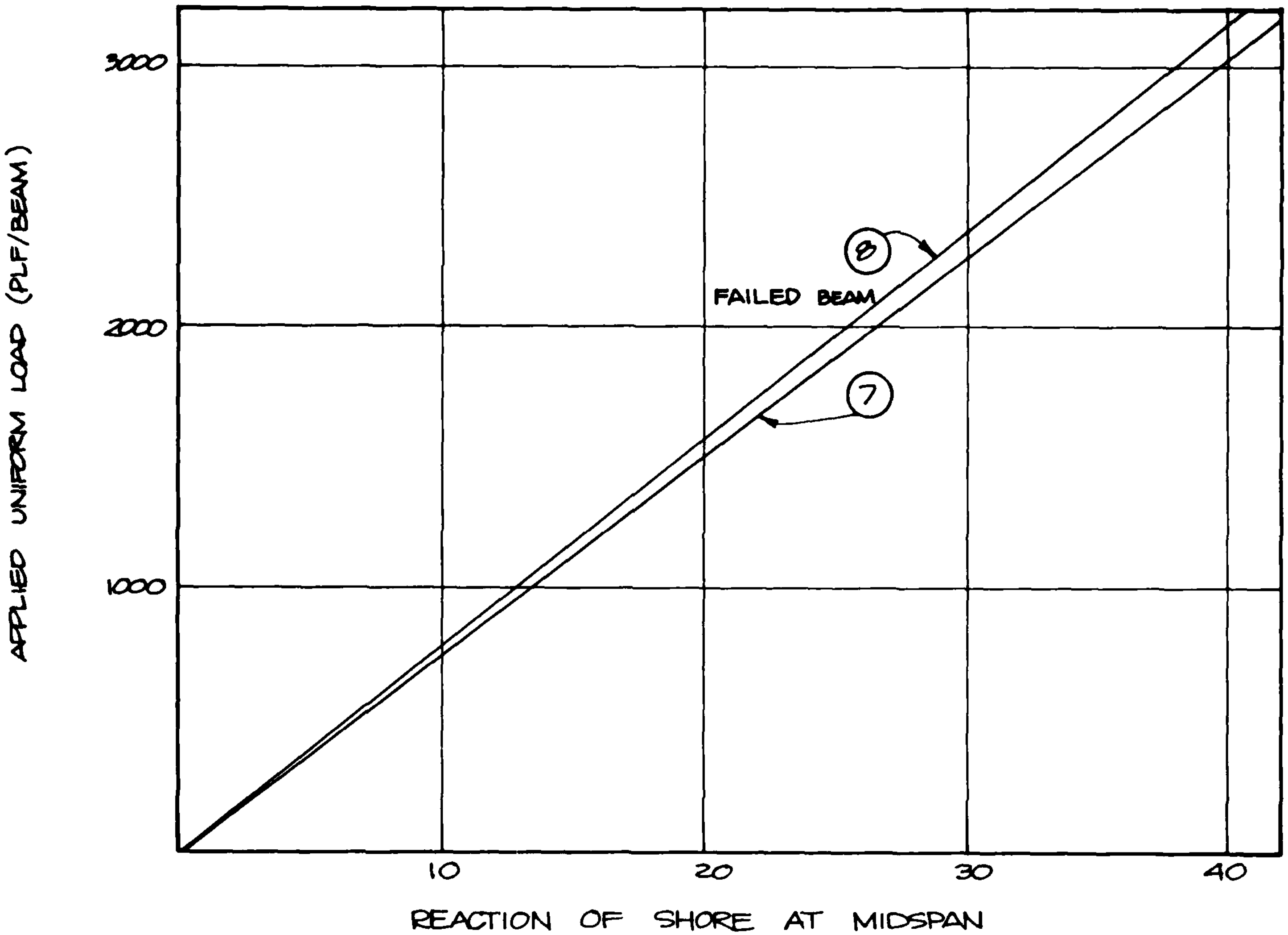


Fig. 3-37. Load vs Shore Reaction at Midspan.

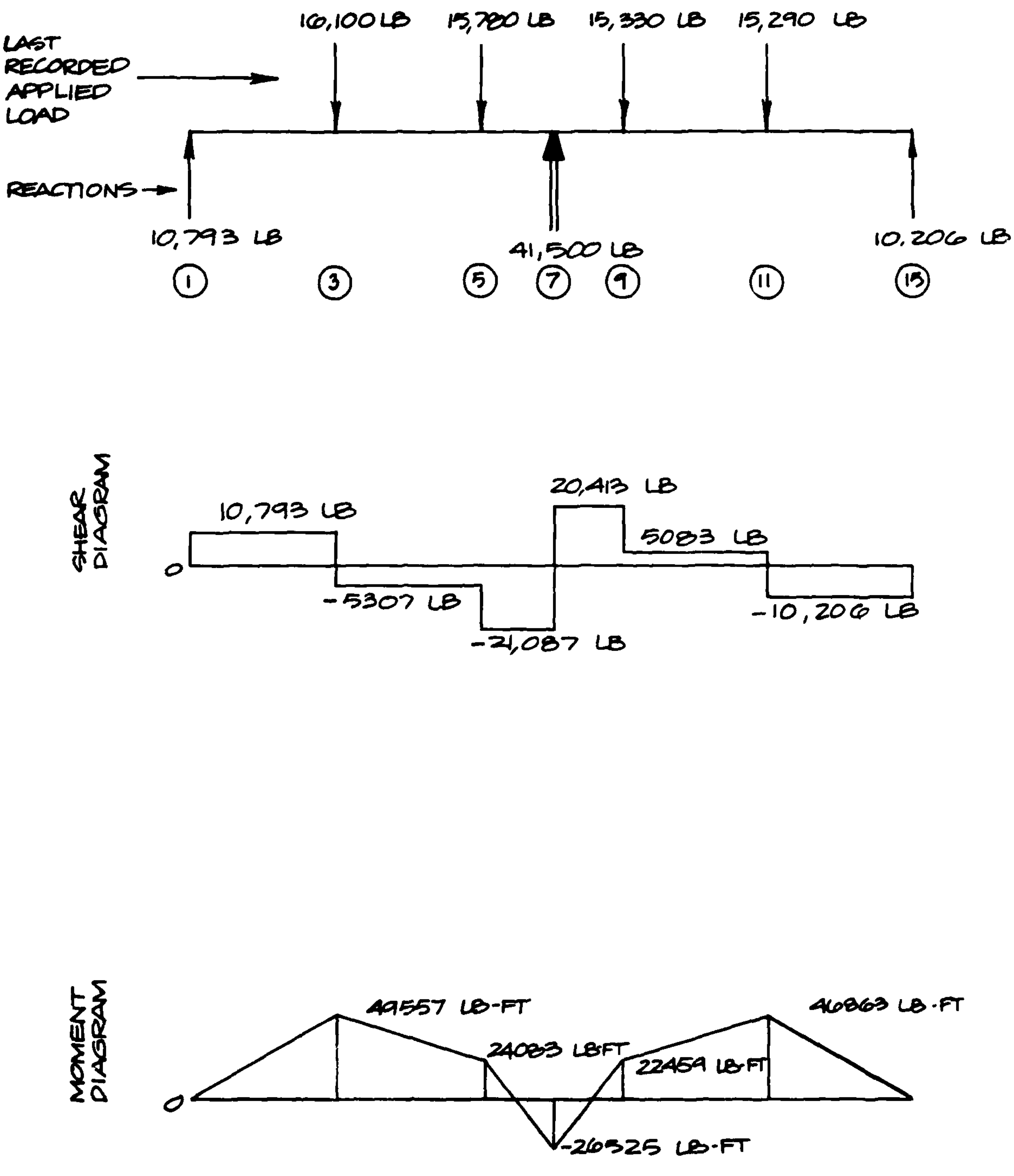


Fig. 3-38. Load, Shear, and Moment Relationships for the 3-1/8" by 18" Glulam Test Specimen Shored at Midspan With a 14-Inch Long Bearing Plate, for the Beam That Did Not Fail.

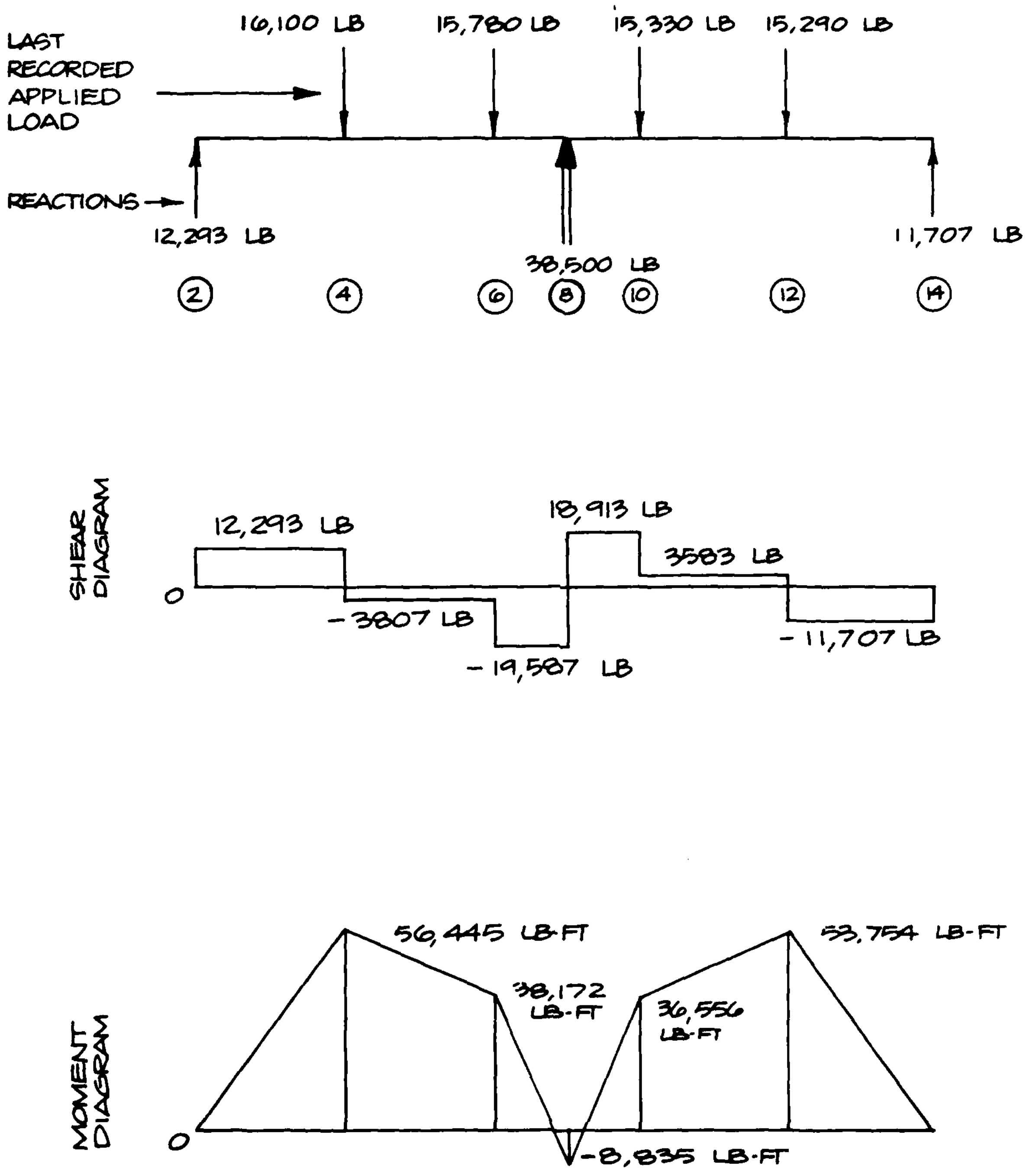
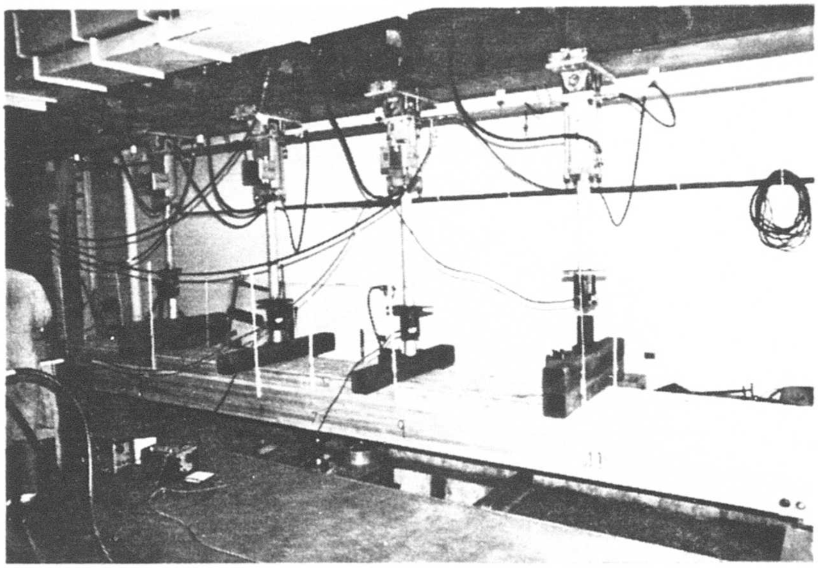


Fig. 3-39. Load, Shear, and Moment Relationships for the 3-1/8" by 18" Glulam Test Specimen, Shored at Midspan With a 14-Inch Long Bearing Plate, for the Beam That Failed.

A



B

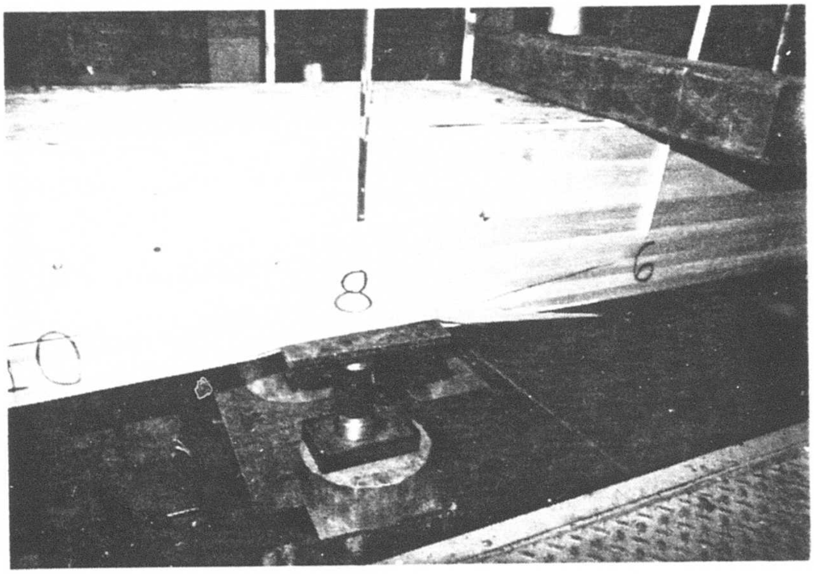
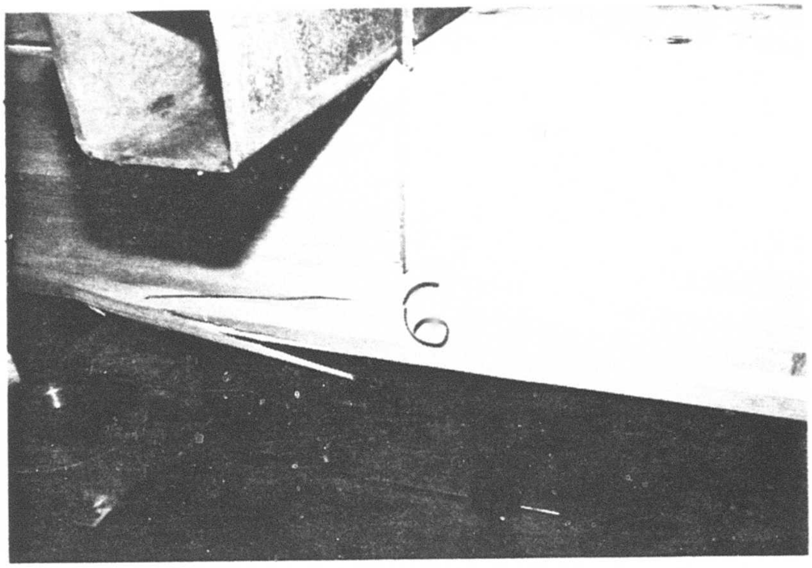
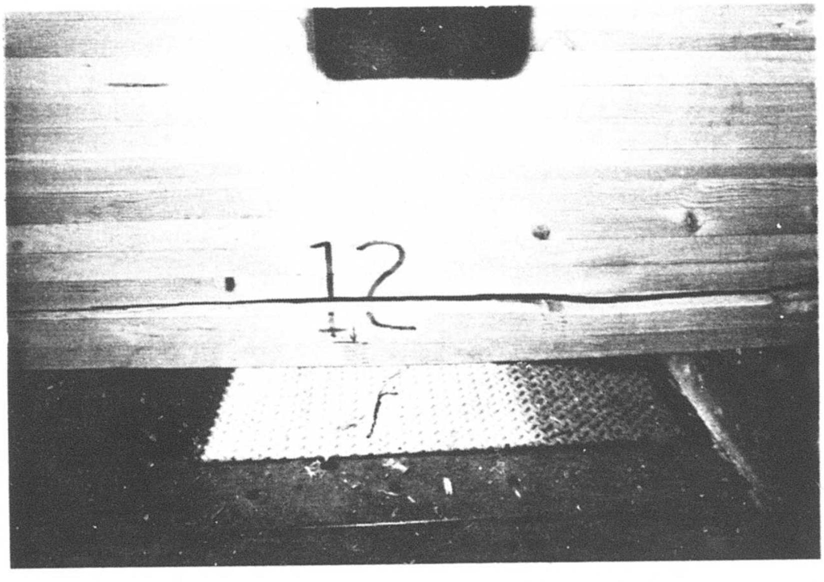


Fig. 3-40. Pre- and Post-Test Photographs of Test Specimen.



A



B

Fig. 3-41. Posttest Photographs of Test Specimen.

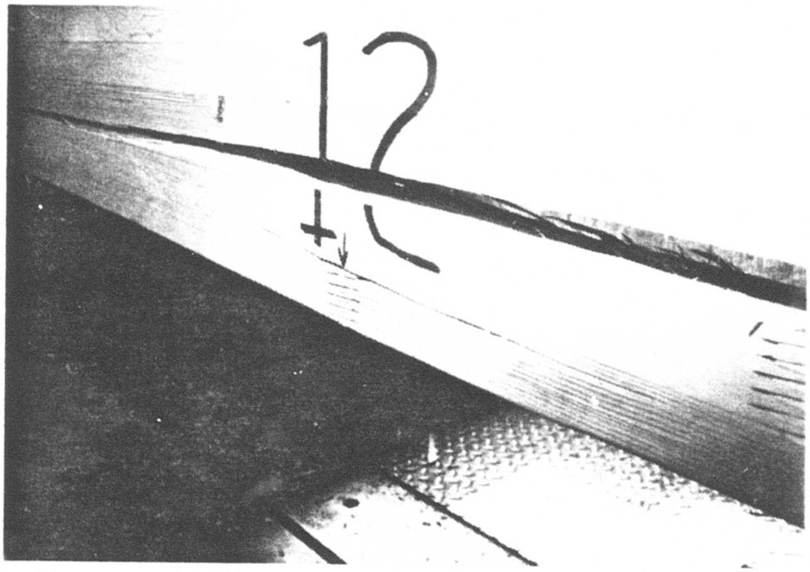


Fig. 3-42. Flexural-Finger Joint Failure in the Tension Laminate on the Failed Beam.

RESULTS AND CONCLUSIONS

The results for each glulam tested are shown in Table 3-1. The base cases for both glulam configurations tested performed as predicted. The factor of safety, FS, was determined by dividing the allowable design load into the ultimate failure load (columns 6 and 4, respectively), resulting in an FS of 1.51 for the 6-3/4" x 16-1/2" glulam beam and an FS of 1.64 for the 3-1/8" x 18" glulam beam. The minimum factor of safety for a typical glulam is 1.3. (This number is based on the 95% exclusion value; i.e., 5% of the members can be expected to have an FS less than 1.3, while 95% will have an FS greater than 1.3.)

Column 5 in Table 3-1 shows the effects upgrading had on the ultimate load-carrying capacity for each of the glulam beams tested. The glulams shored with 10-inch long plates performed better than the ones with 14-inch long plates. The shorter plates produced larger deflections due to crushing-severing of the wood fibers. The larger midspan deflections with the smaller plates had a twofold impact on the flexural and shearing forces in the glulams:

1. Negative moments at midspan were decreased, resulting in increased positive moments at the quarter spans, in a manner similar to the moment redistribution concept used in plastic or limit design.

2. Bearing forces and shear stresses at midspan were also reduced because of the increased amount of settlement.

The 14-inch bearing plate at midspan was selected on the assumption that the glulam beams would remain elastic to failure with the midspan shore receiving 60% of the applied load and the two end supports each receiving approximately 20% of the applied load. The ends of the beam were supported with 5-inch long beam seats, and the midspan bearing plates at 14 inches were sized to be approximately in proportion to the anticipated elastic reactions.

The 10-inch bearing plates were purposely undersized in order to create localized bearing failures at the midspan shores, thus redistributing the shear and moment away from the highly stressed section at midspan. As a rule of thumb, the center shore bearing plate length should equal the sum of the end support plate lengths.

It can be concluded that glulam beams can be successfully shored at midspan to carry approximately three times the simply supported ultimate load, provided the bearing plate for the midspan shore is equal in area to the sum of the end plate areas.

In order that the data developed in this test program may be more accurately included in the prediction methodology, a supplemental investigation based on tests by the Forest Products Laboratory, and including the three base case tests conducted herein, was conducted and is reported in Appendix D.

TABLE 3-1. SUMMARY OF RESULTS

1	2	3*	4**	5	6	7
Glulam Size (in inches)	Method of Upgrading	Total Failure Load (plf)	Failure Adjustment for Duration of Load Effects (plf)	Upgrading % of Base Case (%)	Allowable Design Load (plf)	Equivalent Failure Load (psi)
6-3/4 x 16-1/2	None; base case	1,872	1,291	100	855	1.19
"	Shored @ midspan with 10-in. long bearing plate	6,867	4,736	367	n/a	4.38
"	Shored @ midspan with 14-in. long bearing plate	6,292	4,339	336	n/a	4.01
3-1/8 x 18	None; base case	1,145	774	100	471	0.54
"	Shored @ midspan with 10-in. long bearing plate	3,337	2,301	297	n/a	1.59
"	Shored @ midspan with 14-in. long bearing plate	3,263	2,204	285	n/a	1.53

* Based upon a 10-year duration of load; adjustment factor = 1/1.6.

** Based upon a 5-minute duration of load.

Section 4
CONCRETE STRUCTURAL CONNECTIONS

INTRODUCTION

This section of the report is a review of the types of concrete connections and connection systems that most directly affect the performance of potential shelter options. With respect to system types, concrete construction can logically be divided into two basic categories -- cast-in-place and precast. Each of these categories is discussed separately below.

CAST-IN-PLACE CONCRETE

This type of construction, as the name implies, consists of concrete transported to the building site in a plastic state, placed into or on top of forms, and vibrated and compacted in and around the reinforcing steel. The nomenclatures used to describe the various construction types are based on the different design philosophies employed. These different design methods may result in a somewhat different appearance and performance characteristic for each system type. However, from the standpoint of an investigation of connection integrity under blast loading, many of these systems have much in common and their differences are, in many cases, technically insignificant.

Reinforced concrete floors or roofs are defined as a slab supported so as to effect a successful, efficient, and economical transfer of the floor or roof loads to the columns, and then to the foundation through the footings. The slab may be supported by reinforced concrete beams and/or girders, masonry or reinforced concrete walls, or directly on the columns. Following is a description and evaluation of the various types of typical cast-in-place concrete slabs and their related connections.

One- and Two-Way Slabs

When a slab has its principal reinforcement in one direction and is supported on two sides by beams or walls, it is defined as a one-way slab. If the principal reinforcement is in both directions, it is a two-way slab, and therefore, one-way slabs are really only special cases of two-way slabs. If the ratio of the long side to the short side of the slab is 2 or greater, the slab is probably designed as if all the bending is in the short direction, hence a one-way slab. If this ratio is less than 2, bending must be assumed in both directions, and the slab is designed as a two-way slab. Obviously, the use of this ratio is only for the purpose of defining the type of system and, in fact, the ACI Building Code (Ref. 6) does require investigation of the design of the reinforcement in the long direction of a slab supported along its four edges even though the ratio of long side to short side may be greater than 2 (Ref. 7).

The beams, columns, and, many times, even the walls are cast monolithically with the slabs. For this reason, the slabs can, and usually are, designed to be continuous over the beam supports, with the end spans tied into the edge beams and/or walls for moment resistance. Typical connections associated with these types of slabs are shown in Figure 4-1. This degree of "fixity" at supports provides a certain amount of redundancy in the system, which permits a redistribution of stresses under severe loadings. These slab systems perform well structurally as shelters when properly upgraded. The MILL RACE high explosive test in 1981 contained a basement structure with the floor above consisting of two bays of a 6-in. thick, two-way slab, shored, and subjected to a 40 psi blast environment. The performance of this shelter area was quite good, with no severe structural damage noted (Ref. 8). A number of tests on one-way slabs, both shored and unshored, have been conducted by Scientific Service, Inc. (Refs. 2 and 9), and the U.S. Army Waterways Experiment Station (Ref. 10).

One-Way Joist Slabs

One-way joist construction is a special case of the one-way slab. Since the concrete located between the neutral axis and the tension face of a solid reinforced concrete slab does not significantly contribute to the flexural strength, but is effective in resisting portions of the shearing stresses, it is possible, under certain

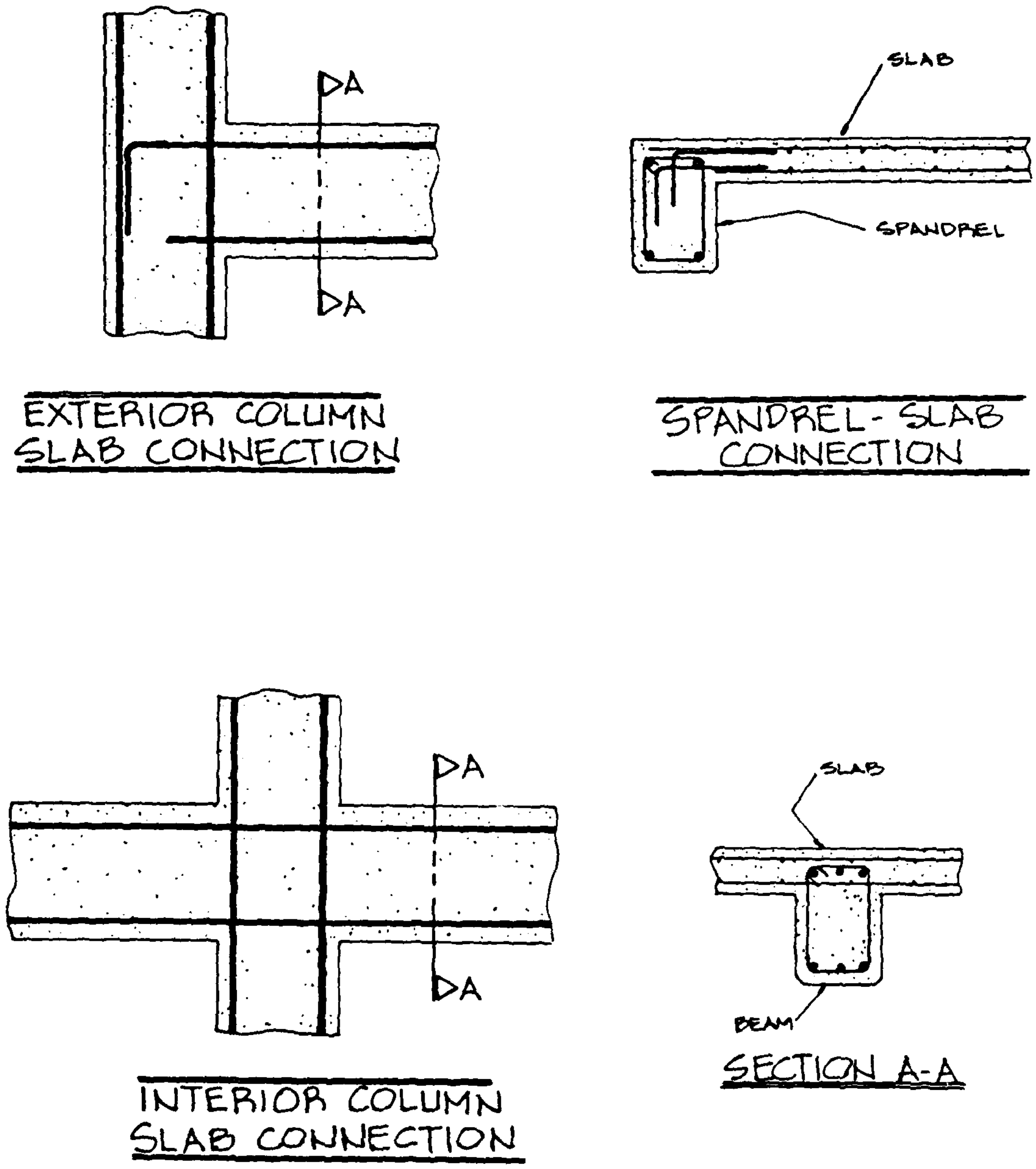


Fig. 4-1. Connections for One- and Two-Way Slabs.

load and span conditions, to eliminate a large portion of the concrete on the bottom side of the slab, leaving only ribs or joists, the bottom of these joists corresponding to the bottom of the solid slab. This configuration saves concrete and thereby reduces the weight of the slab. The reinforcing steel, which was distributed rather evenly throughout the solid slab, is now concentrated in each of the joists; less reinforcement is required, however, since the dead weight of the system has been reduced.

Since the formwork for this type of construction is more complicated and thereby more expensive, the construction industry has developed standard size forms that may be rented or purchased. Standard form sizes typical for this type of construction result in void spaces between joists of either 20 or 30 in. wide, and may be obtained in depths of 6 to 20 in. (Ref. 11).

The one-way joist system is cast monolithically and has much of the redundancy of the one-way slab discussed above, and if properly upgraded, the joists and beams and their related connections would be expected to perform well. This type of construction is shown in Figure 4-2. However, a question remains with respect to the slab portion between the joists. These slabs are typically 3 to 4½ in. thick and are minimally reinforced. Tests 'ave been conducted on slabs of this thickness and span, but supported on all four edges, and the results of these tests indicated good performance primarily because of the presence of membrane action. It is doubtful, however, if this membrane action would be as effective in a slab supported on only two edges, and further investigative effort in this area is required.

Flat Slabs and Flat Plates

A flat slab is a concrete slab reinforced in two directions so that it transfers its loads directly to supporting columns; i.e., it has no beams or girders to transfer the loads to the columns, such as one- and two-way slabs. In order to better resist the stresses concentrated immediately surrounding each column, this type of construction typically uses a flared column capital and often has a thickened slab, or drop panel, around the column. The configuration of these connections is shown in Figure 4-3.

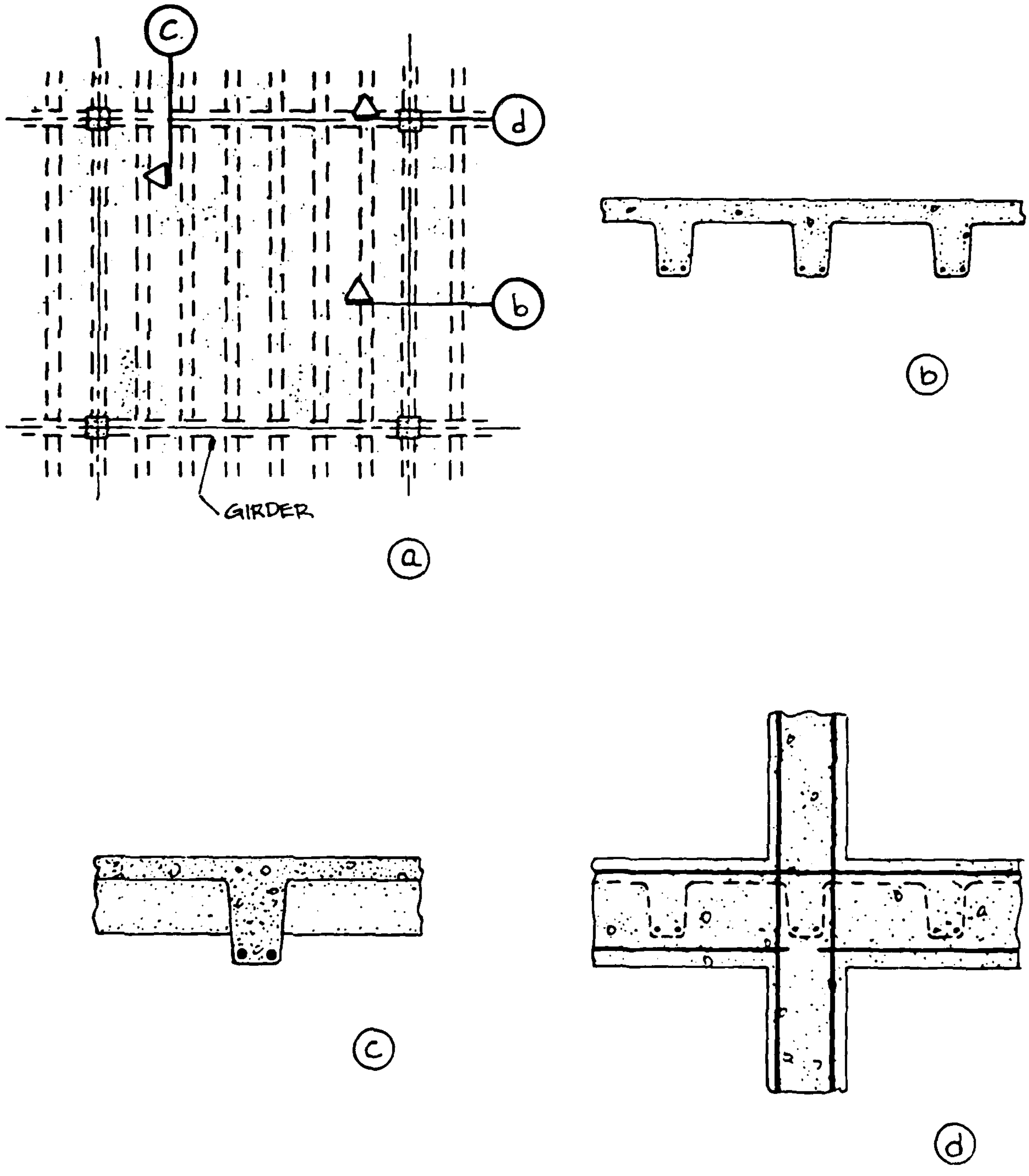


Fig. 4-2. One-Way Reinforced Concrete Joists.

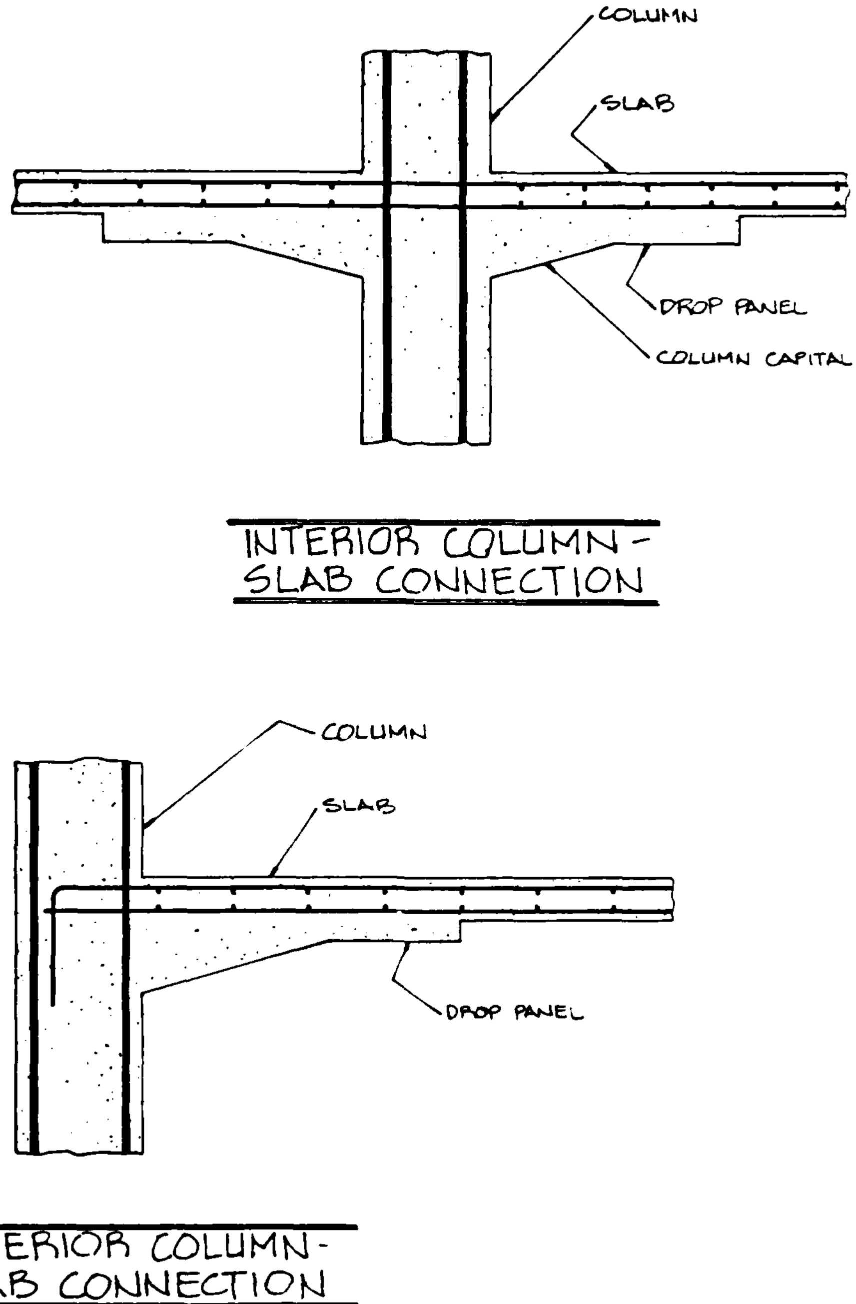


Fig. 4-3. Flat Slab Connections.

A flat plate is also reinforced in two directions and transfers its loads directly to the columns, but is constructed without column capitals or drop panels. These connections are illustrated in Figure 4-4. Since no special accommodations are made at the column, either by thickening the slab or by the addition of capitals, the shearing stresses at these locations limit the load for which flat plates are feasible. However, they are extensively used because of their flat uninterrupted ceilings, particularly as ceilings in areas where partitions are to be installed. At the present time, a considerable number of flat plates use post-tensioned reinforcement. This type of construction will be discussed in more detail below.

As was the case with the one- and two-way slabs, flat slabs and plates are typically cast monolithically with the columns, and sometimes the walls. They are designed to be continuous over the column supports, and their end spans are tied into the walls and/or edge beams providing moment resistance. This type of design and construction again provides a certain amount of redundancy in the system that permits stress redistribution prior to collapse under severe loadings. When upgraded properly, the flat slab type of construction would be expected to perform well as a shelter, and the connections would not require any special consideration. Full-scale field tests of this type of construction subjected to 40 psi have been successfully conducted (Ref. 8).

Flat plate construction, however, because of the design parameters previously outlined, would not be expected to perform quite as well. Under severe loading, a shear failure would be anticipated adjacent to and around the columns; i.e., the column would punch through the slab. This column/slab connection would probably be the critical failure mechanism for this type of system even with the slab portion properly upgraded. This type of failure mode is clearly indicated in a nine-bay, one-quarter scale model test conducted by the USAE Waterways Experiment Station in February, 1982 (Ref. 12). Accordingly, in order to consider flat plates as viable shelter options, particularly in risk areas, an upgrading methodology for these critical areas adjacent to the column supports must be developed and verified by test.

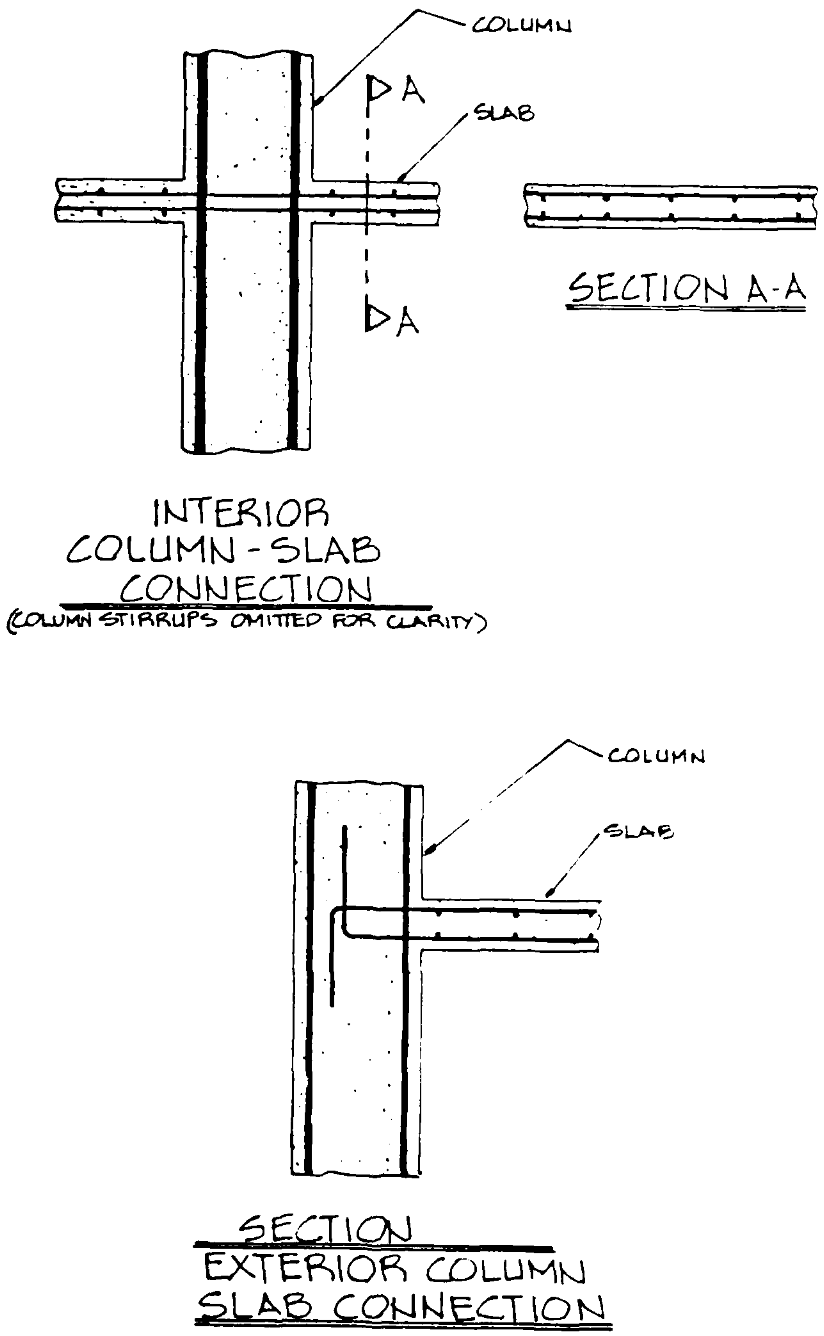


Fig. 4-4. Flat Plate Connections.

Waffle Slabs

Waffle slab construction consists of rows of joists at right angles to each other with solid heads at the columns. For design purposes, waffle slabs are considered flat slabs, with the solid heads at the columns performing the same function as drop panels. The economic basis for this type of construction is the same as was outlined above for one-way joist slabs; i.e., a reduction in concrete and weight, and a corresponding reduction in required reinforcing steel. As with the flat slab, the principal reinforcement is in both directions, but in this case located in the joists, and the slab is cast monolithically with the columns and walls. The standard forms for waffle slabs are either 30 in. or 19 in. square, the former providing 36 in. on center joist spacing with 6-in. wide joists, and the latter providing 24 in. on center joist spacing with 5-in. wide joists. The 6-in. joists have standard depths of 8 to 20 in. and the 5-in. wide joists, 4 to 12 in. (Ref. 10).

This slab system would be expected to perform well as a shelter, similar to the flat slab; if properly upgraded, the connections would not require special consideration. The slab portion between the joists, as with the one-way joist slab, typically ranges from 3 to 4½ in. thick. However, unlike the one-way joist, test data indicate that this thin slab section, because of the membrane action developed by support at all four edges, should not be a detriment for use as a shelter. Tests have been conducted by the U. S. Army Waterways Experiment Station (reports unpublished) on full-scale waffle slabs and small sections of the thin slab sections from waffle slabs. This type of construction is shown in Figure 4-5.

Post-Tensioned Slabs

In the last 15 years in the United States, post-tensioned construction has increased some 350%, primarily because of the desire to obtain reduced structural system depth and longer spans, at an economical cost (Ref. 13). At this point, a very brief description of post-tensioning theory and construction methods is in order for those who are unfamiliar with this type of construction. The strength properties of concrete are such that it is a good material in compression, but relatively weak in tension; i.e., the compressive strength is approximately ten times the tensile strength. In the design of concrete slabs using normal reinforcing steel, the steel is located in areas where tensile stresses are anticipated, thus utilizing the concrete in

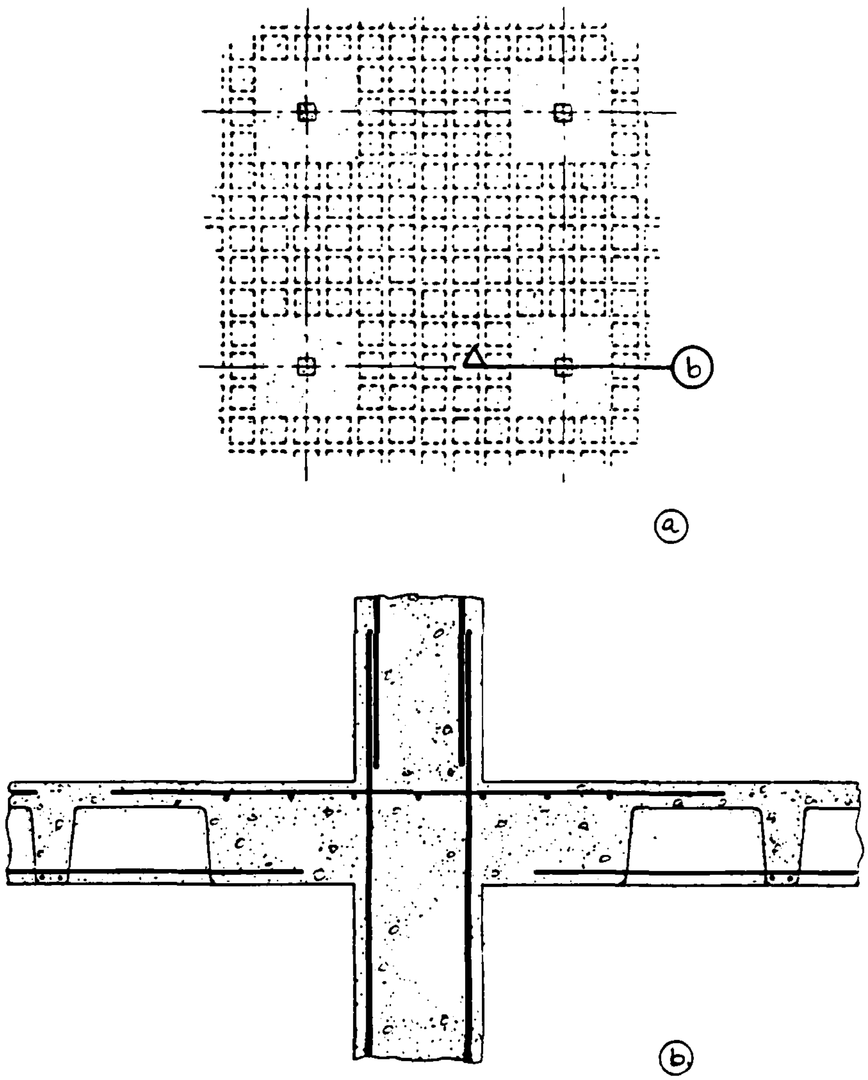
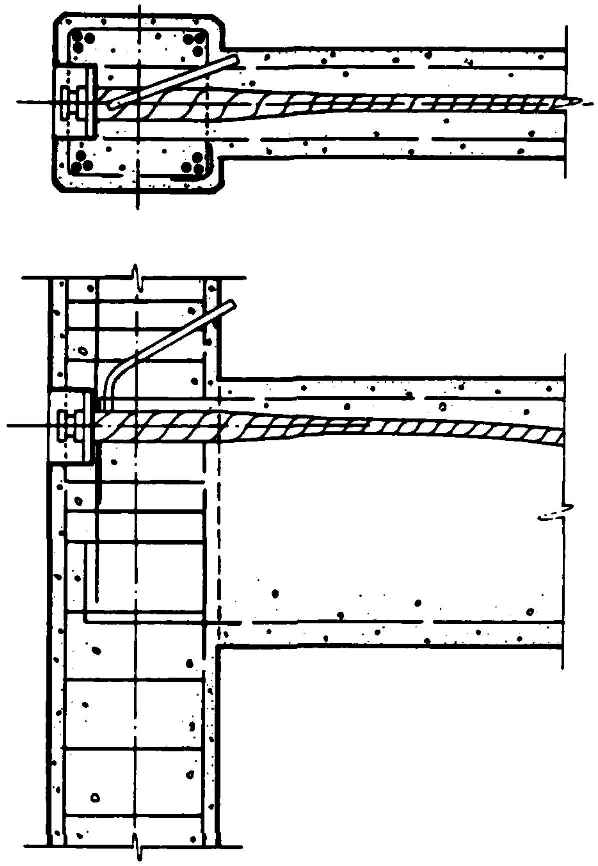


Fig. 4-5. Waffle Slabs.

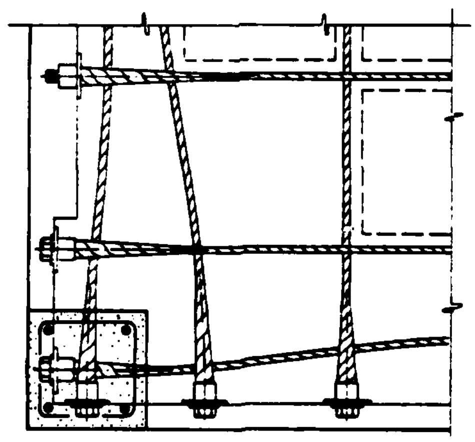
compression and the steel for tension. Post-tensioning is a method by which compressive forces are induced into the slab by elongating wires or strands, called tendons, placing concrete around the tendons, allowing the concrete to attain its desired strength, and securing the tendons by filling a duct that surrounds the tendon with grout and anchoring the tendons at their ends (bonded post-tensioned), or by anchoring the tendons at their ends only (unbonded post-tensioning).

In post-tensioned slab construction, the load carrying capability of the slab is obtained by placing a series of tendons accurately on the slab formwork prior to placement of the concrete. These tendons may be several bays or more in length, spaced at specific intervals, and are typically in a parabolic profile to accommodate both the negative and positive moments in continuous spans. If the tendons are to be "unbonded", they are greased and paper wrapped or plastic covered, and if later to be "bonded", placed in a flexible duct, in order that they do not initially bond and may slide easily through the slab to accommodate the later stressing operation. Once the concrete is placed and reaches sufficient strength, each tendon is elongated the calculated amount, thus tensioning each to a design stress. To hold the tendon at this final stress level, anchoring devices with steel jaws are utilized at each end of the tendon. If the tendons are to be "bonded", the duct is pumped full with grout. These anchoring devices are later covered with concrete for fire proofing and corrosion protection. Typical tendon anchorage details are shown in Figure 4-6.

There are inherent problems in upgrading and utilizing post-tensioned slabs as viable shelter options. When tendons are unbonded and a tendon failure occurs in one span, many spans may be affected. As stated in the "Post-Tensioning Manual" (Ref. 13), "A catastrophic loading such as might occur from an explosion or a severe earthquake which resulted in a failure in one bay of a beam or one-way slab with unbonded tendons could result in a progression of the failure throughout all bays of a multi-bay building." These types of progressive collapses have occurred in the past. One notable example is illustrated by the parking structure at Bailey's Crossroads, Fairfax County, Virginia, which, in 1973, totally collapsed when debris fell on a portion of the slab from an adjacent high-rise construction collapse (Ref. 14). Building code revisions have, to a small degree, addressed this problem in one-way



ANCHORAGE AT COLUMNS / WALLS



TENDON ANCHORAGE AT CORNER COLUMN

Fig. 4-6. Post-Tensioned Connections (Ref. 13).

unbonded post-tensioned slabs by requiring secondary means of carrying loads; i.e., additional bonded tendons or reinforcing steel. It is the writer's opinion that these code requirements are inadequate, and when and if improved, should be extended to two-way slabs. Two-way slabs are now excluded from this revision on the assumption that a loss of load-carrying capability would occur in only one direction, an assumption that is highly speculative. This code change was instituted in 1976 and probably was not implemented in building design methodology to any great degree until at least 1978. Accordingly, the great majority of existing structures that would now be considered for use as shelters do not have the advantage of even this limited code provision.

A review of the current literature on failure analysis investigation indicates that post-tensioned construction, particularly unbonded, has a relatively poor performance record. Significant problems that have been reported include loss of precompression of the slab due to creep and shrinkage of the concrete, a condition that is approximately three times greater in unbonded post-tensioned slabs than in conventionally reinforced slabs, and corrosion of end anchorages, which are highly susceptible to stress corrosion if not adequately protected.

In order to accommodate the plastic deformation or creep of the concrete slab with time, as well as from shrinkage and thermal volume changes, Ref. 13 recommends a number of connection details that minimize restraint; one such detail is shown in Figure 4-7. Although these suggested details serve this purpose, they further degrade connection integrity with respect to blast survival, and accordingly, use as a shelter.

Post-tensioned slabs are currently used in many structures, and it is expected that their use will increase; therefore, it is necessary to include this type of construction, if possible, for consideration as a shelter option. At the present time a slab of this construction would not be considered a viable candidate for shelter upgrading. An investigative effort is required in order to identify the specific problem areas and to develop and test possible upgrading procedures.

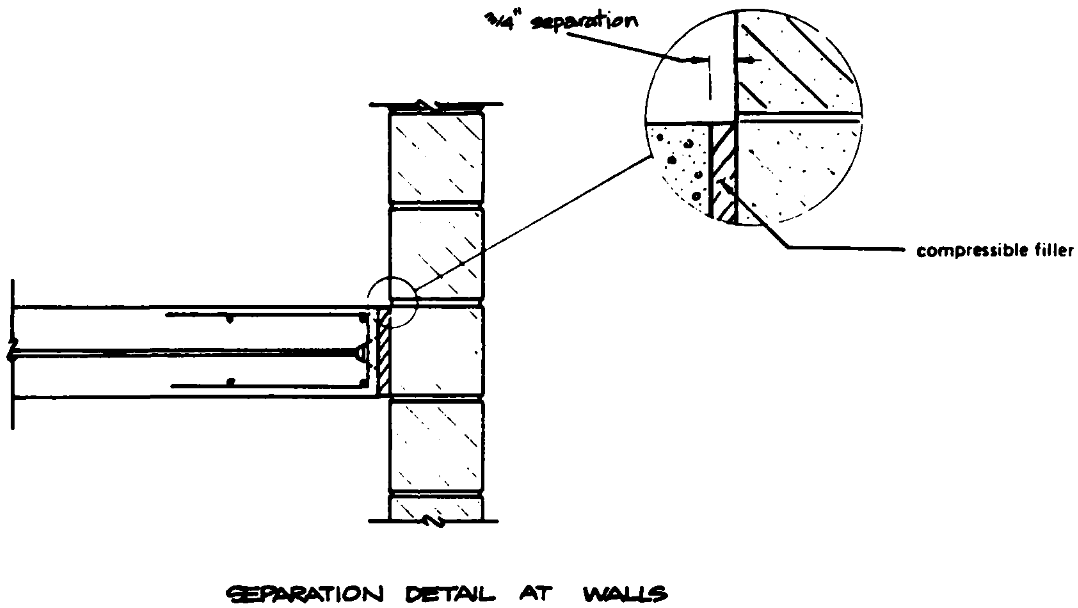


Fig. 4-7. Post-Tensioned Connections (Ref. 13).

PRECAST CONCRETE

Precast concrete is defined as concrete that is cast in some location other than its final position in a completed structure. The location may be a plant that specializes in the manufacture of particular elements, or actually the building site, where a contractor may use temporary forming methods to produce the elements on a one time only basis. In either case, the common characteristic of precast concrete is that the individual elements, or building components, must be located in their final position, and connected and/or secured to supports, footings, or each other, in order to perform as designed. Static and dynamic load tests of buildings, or portions of buildings, as well as the investigation of building failures, have shown that in many cases the connections associated with precast concrete are the weak link in this type of construction.

In the construction of a total building, precast concrete components may be used in combination with structural steel and/or cast-in-place concrete, or used to construct practically the entire structure. These components may be structural precast (i.e., they perform a function in the building necessary to its structural integrity), or architectural precast, or a combination of both. The precast components may be pilings, single or multi story columns, beams and girders, single and double tees, hollow-core or solid slab floor or roof members, or wall panels consisting of solid units, single or double tee and hollow-core members, or insulated sandwich units. The wall panels may designed to be either load bearing or non load bearing components (Ref. 15). Other types of precast components are curtain wall cladding, stairway units, sun shades, and spandrel beams. It is obvious from this partial list that the uses of precast concrete, and the shapes and configurations in which it may be obtained, are limited only by economics and the imagination of the architect. However, for the purpose of evaluating buildings constructed of this material as options for use as shelters, we need only concern ourselves with components used as the main structural elements; i.e., the elements required to support the intended design loads, and subsequently when upgraded, the blast loading. Accordingly, we will confine the investigation to elements such as walls, floors and roofs, columns, beams and girders, and their related connections.

It is recognized that many of the architectural precast components mentioned above, such as wall cladding and sun shades, could provide a serious hazard to the sheltered population with respect to the translation of debris; an investigation of this problem, however, is not within the immediate scope of this section of the report.

The primary reinforcement in precast concrete structural components may be mild steel reinforcing or prestressing strands. However, unlike the required differentiation between mild steel reinforcing and post-tensioning in cast-in-place concrete slabs, both of these reinforcing methods perform very similarly when loaded to failure loads (Refs. 1, 2 and 9). The types of connections used for each reinforcing method are the same and are independent of which design method is used. Although the design methodology between prestressing and post-tensioning is very similar, the construction methods are different. Prestressing results in fully bonded strands that do not depend on end anchorages or grouting for integrity, and the cutting or destroying of one strand at a location along its length does not result in loss of prestress, or load-carrying ability, the full length of the strand.

Since precast concrete structural components of identical configuration may be used in constructing different building elements, they will be combined for evaluation with respect to their use in the building; i.e., floors and roofs, walls, columns, and beams and girders.

Floors and Roofs

The components that are used in floors and roofs are described below and illustrated on Figure 4-8 (Refs. 15 and 16):

> **Solid Flat Slabs** - These are usually fabricated in depths of 3 to 6 in., and may be reinforced with mild steel or prestressed. The width of the units is restricted by shipping and the width of the precaster's casting beds, but typically varies from 8 to 12 ft. These slabs would typically be used in spans of from 13 to 24 ft, and are usually covered with a structural reinforced concrete topping. See Figure 4-8(a).

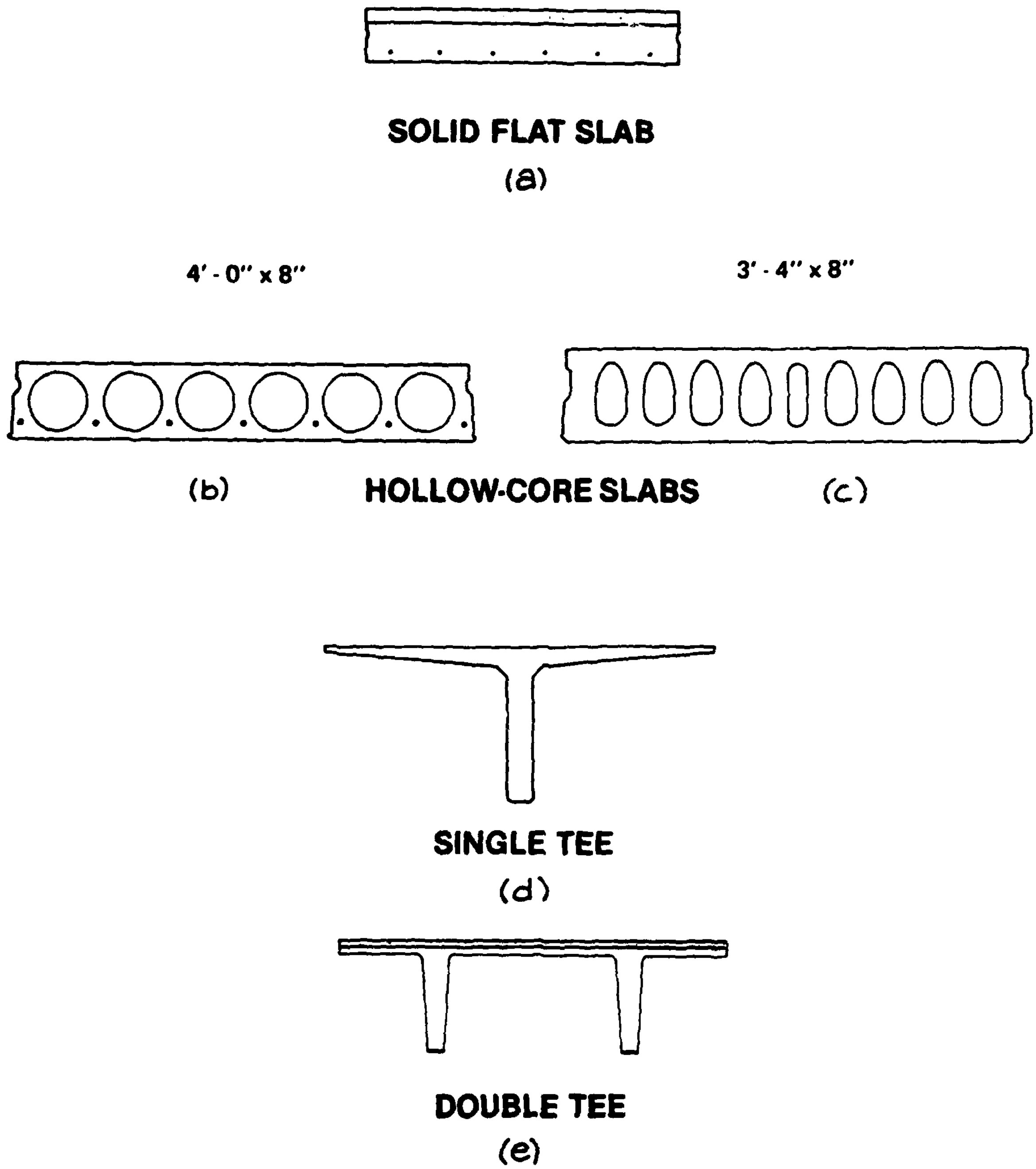


Fig. 4-8. Floor and Roof Slabs (Ref. 16).

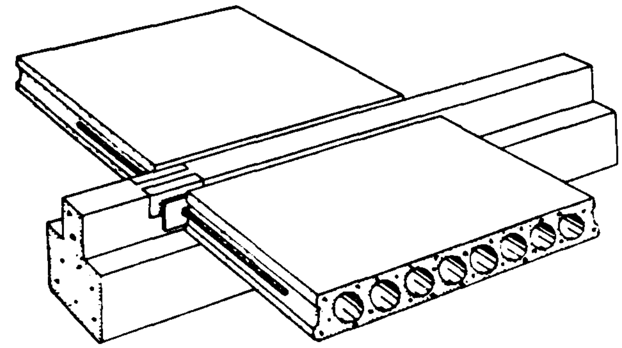
Hollow-Core Slabs - Precast prestressed slabs are manufactured by commercially franchised processes using specialized forming machinery. Six principal processes are used in the United States to produce slab widths from 2 to 8 ft, depths from 4 to 12 in., and core configurations such as round, rectangular, or elliptical. They may be installed side by side in a floor or roof, or positioned up to 3 ft apart with the space in between spanned with metal decking. Typical spans for these units range from 18 to 42 ft. See Figure 4-8(b)(c).

Single Tees - The configuration of these prestressed units, as the name implies, consists of a horizontal slab, or flange, with one vertical stem located at the mid-width of the flange. Single tees vary in width from 6 to 12 ft, and in depth from 16 to 48 in. The flange is typically a minimum of 2 in. thick at the outer edges and increases in depth toward the stem. Single tees usually span greater distances than double tees, up to 120 ft; however, the same economic considerations applicable to double tees are applicable to these units, with the result being somewhat shorter spans in typical use. See Figure 4-8(d).

Double Tees - These prestressed units are plant cast in steel forms and derive their name from their cross-sectional appearance, a horizontal slab with two vertical stems symmetrically spaced. Common variations to this section result when they are cast without the slab portion on the outside of one stem, an "F" slab, or without a slab to the outside of both stems, a channel slab; these modifications, however, do not have any bearing on this investigation. Double tees vary considerably in cross-sectional dimensions, and accordingly, span and load carrying capability. Their width is typically 4 to 12 ft, with depths of from 10 to 41 in. The horizontal slab, or flange, thickness is typically 2 in. throughout its width, but may be increased to 2½ in. to 4 in. because of structural requirements. Spans of up to 90 ft are possible with the deeper sections, but considerations of handling, transporting, and erecting these units result in maximum economical spans of 50 to 60 ft. See Figure 4-8(e).

During the previous discussion of cast-in-place concrete floors and roofs, the fact that this type of construction was cast monolithically with and over its supports, whether beams, columns, walls, etc., resulting in a continuous structural system, was judged to be advantageous with respect to selection as a shelter option. The primary reason for this judgment was the redundancy of that type of construction. That is, the system has multiple load paths, and when overloaded or subjected to unsymmetrical loading for which it was not designed, it has the capability to redistribute the resulting stresses to a stronger load path. The same cannot be said of a non-monolithic, non-continuous system such as a precast concrete floor or roof. Although some types of precast floor and roof systems lend themselves to partially continuous designs, by and large, this type of construction is a simple span. Each individual precast unit, although it may have some ability to transfer loads to immediately adjacent units by weldments or grout joints, must perform independently of the other spans and/or bays.

The normal volume changes occurring in concrete as a result of shrinkage, creep, and temperature must be relieved at the bearing ends in simple span precast concrete components, instead of being distributed throughout the structure, as would be the case in monolithically cast-in-place slab systems. Because of this, as well as the fact that simple span flexural members undergo rotation at the bearing ends, fixed connections are not recommended for precast concrete floor and roof units. The design of these end connections varies with the type of unit. Solid and hollow-core slabs normally rest on bearing pads, usually of felt, asbestos-cement, or hardboard positioned on the supporting walls or beams and have no positive connection, as shown on Figures 4-9, 4-10, and 4-11. Single and double tees are also erected this way, using bearing pads of elastomeric, laminated fabric, or frictionless (Tetrafluorethylene) material (Ref. 17) at each bearing end, or welding at one end, thus achieving a semi-fixed connection at one end, with the bearing pad at the other. Examples of these connections are shown in Figures 4-12, 4-13, and 4-14. This "welded at one end" type of connection is currently widely used, even though Ref. 15 states ". . . axiomatically that the bottoms of double tees and other stemmed prestressed concrete members are never welded at their supports: they are left to 'float' free on neoprene . . . ." In structures older than 15 years, it is not uncommon to find stemmed floor and roof units with both ends welded, a



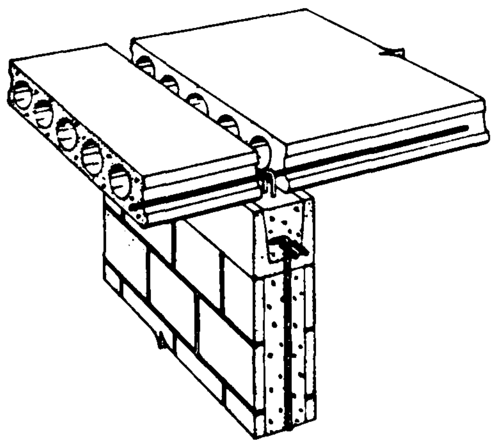
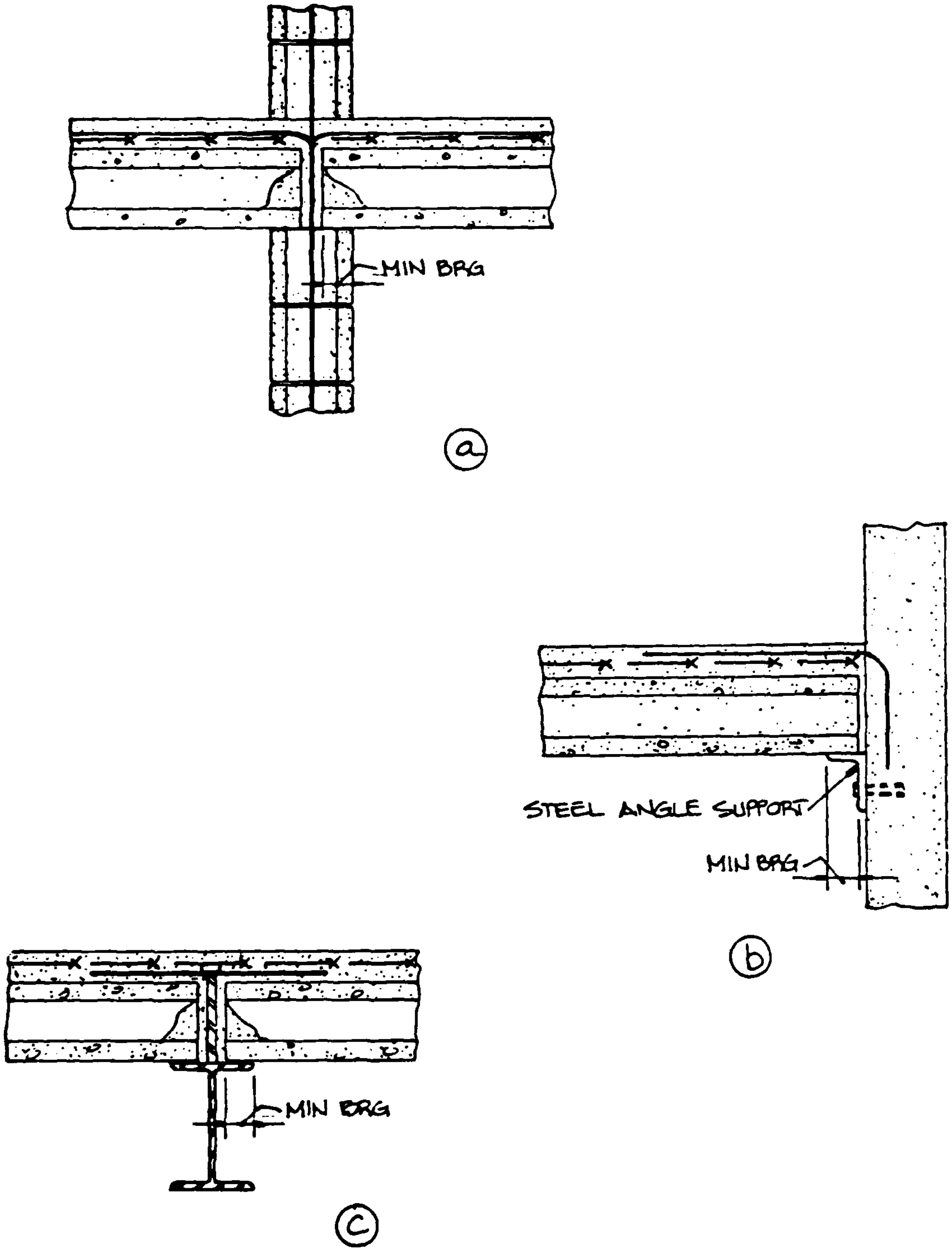


Fig. 4-9. Hollow-Core Slab Connections (Untopped) (Ref. 17).



. 4-10. Hollow-Core Slab Connections (Topped).

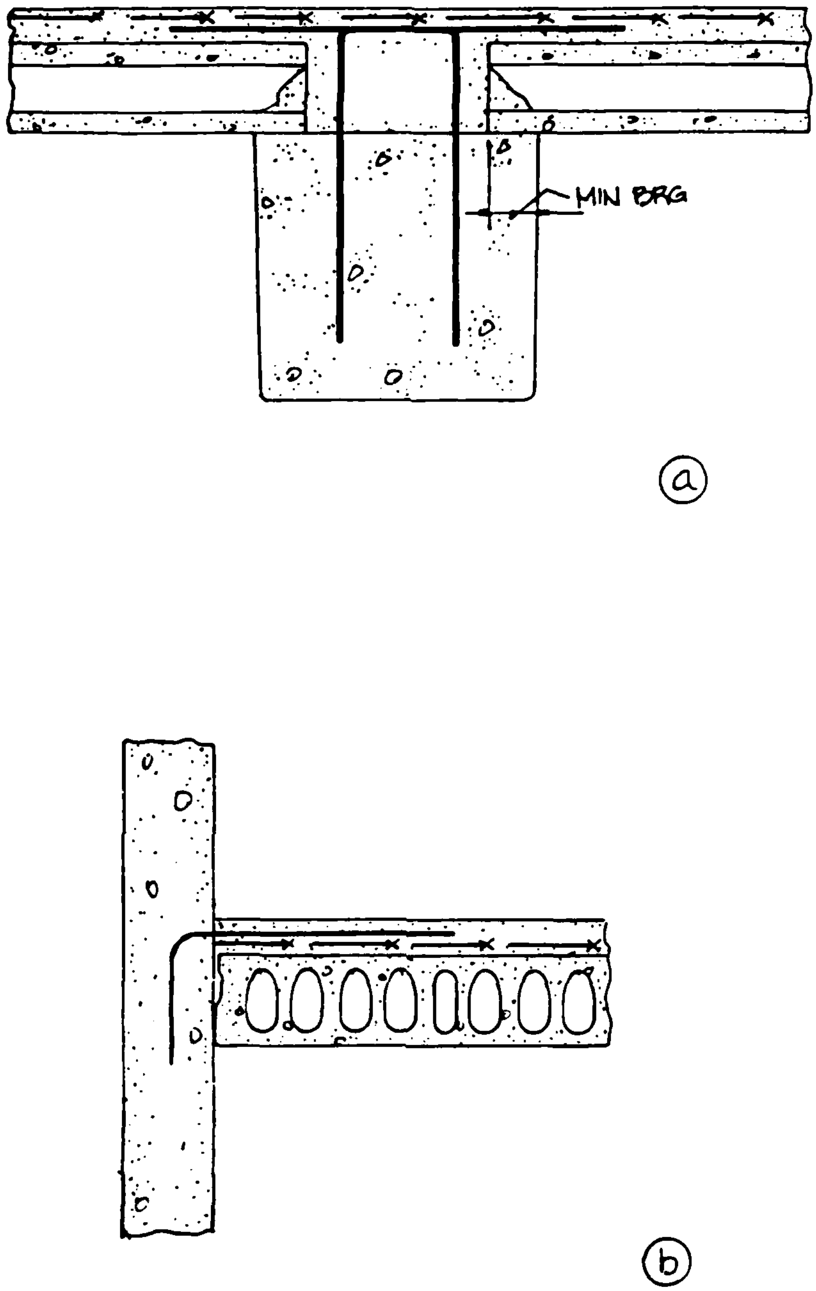


Fig. 4-11. Hollow-Core Slab Connections (Topped).

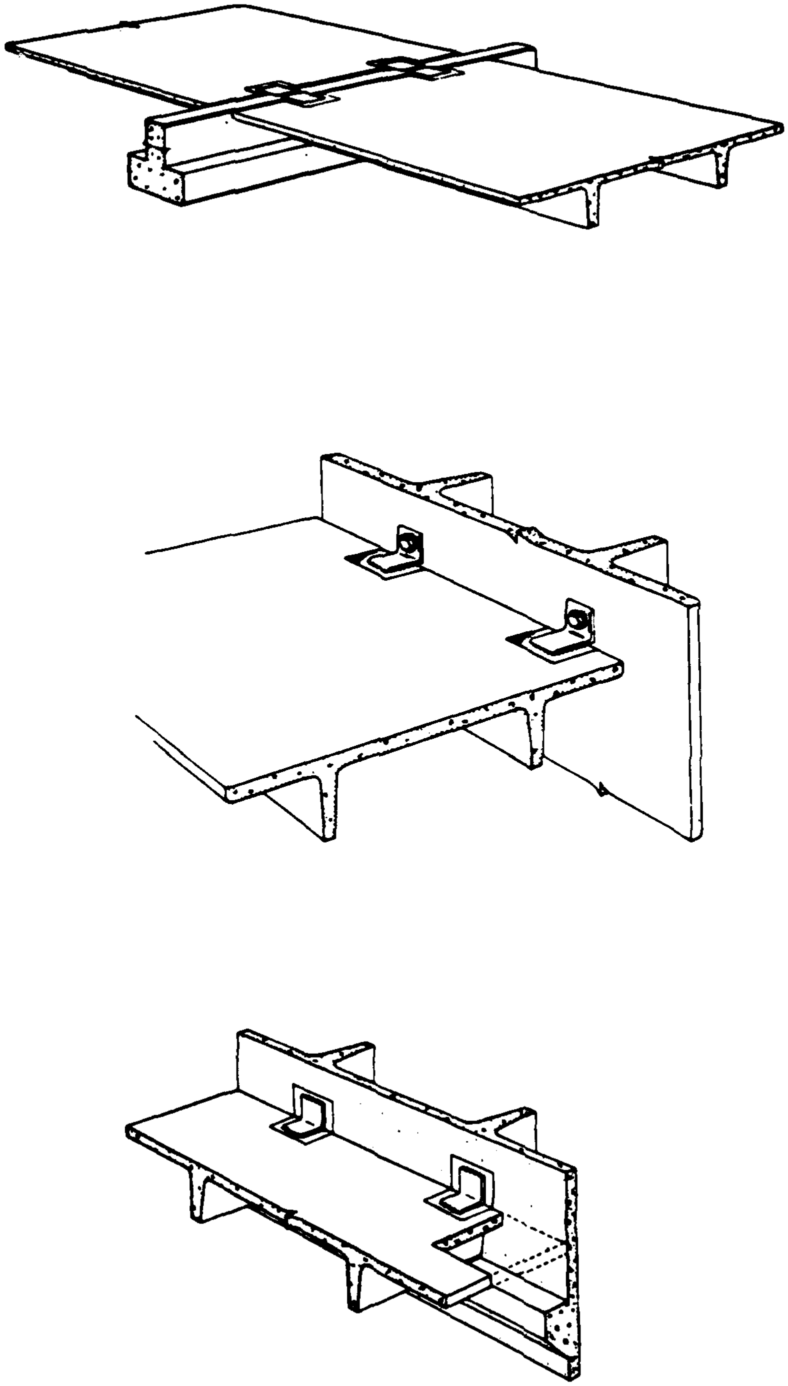


Fig. 4-12. Double Tee Connections (Untopped) Ref. 17).

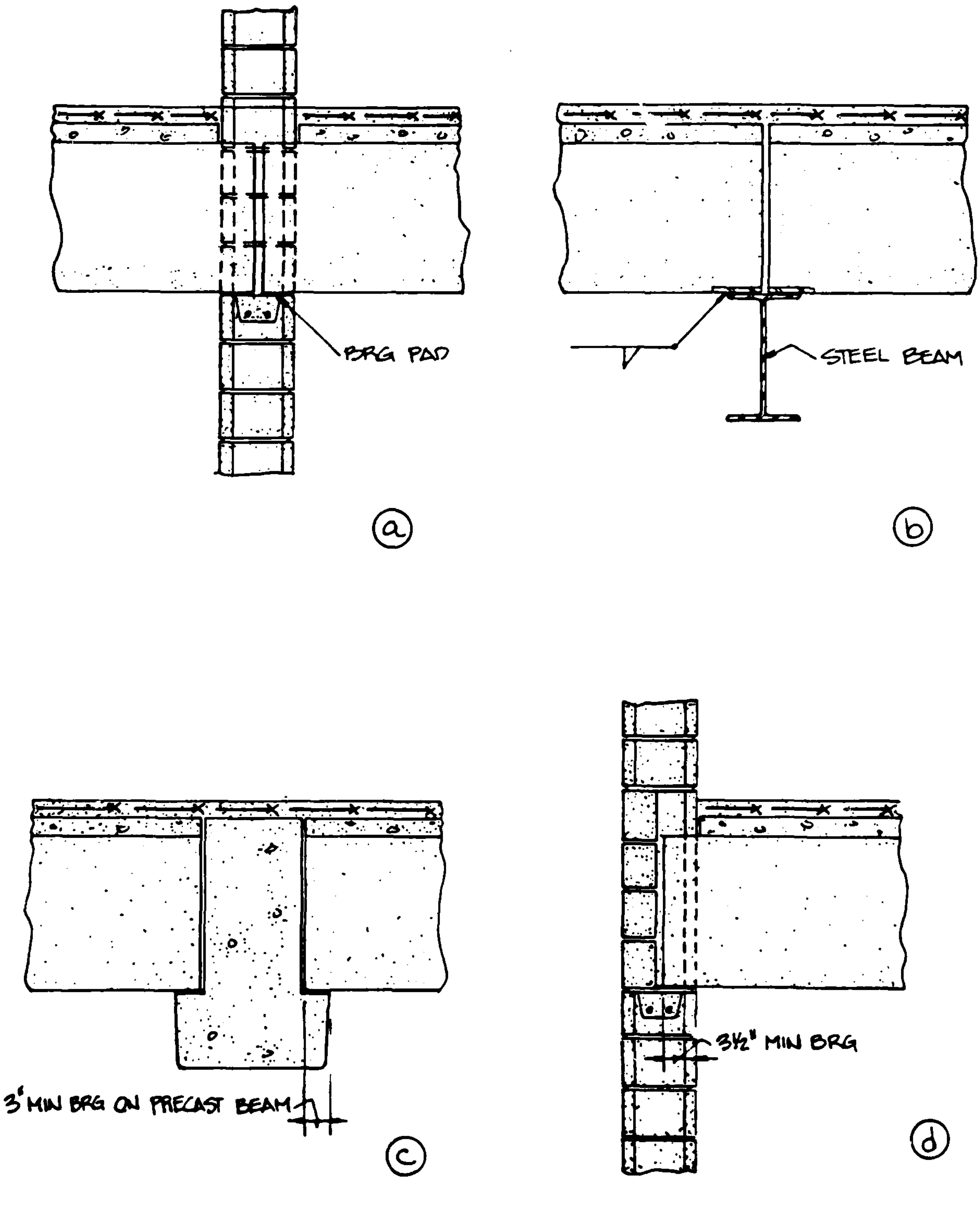


Fig. 4-13. Double and Single Tee Connections (Topped).

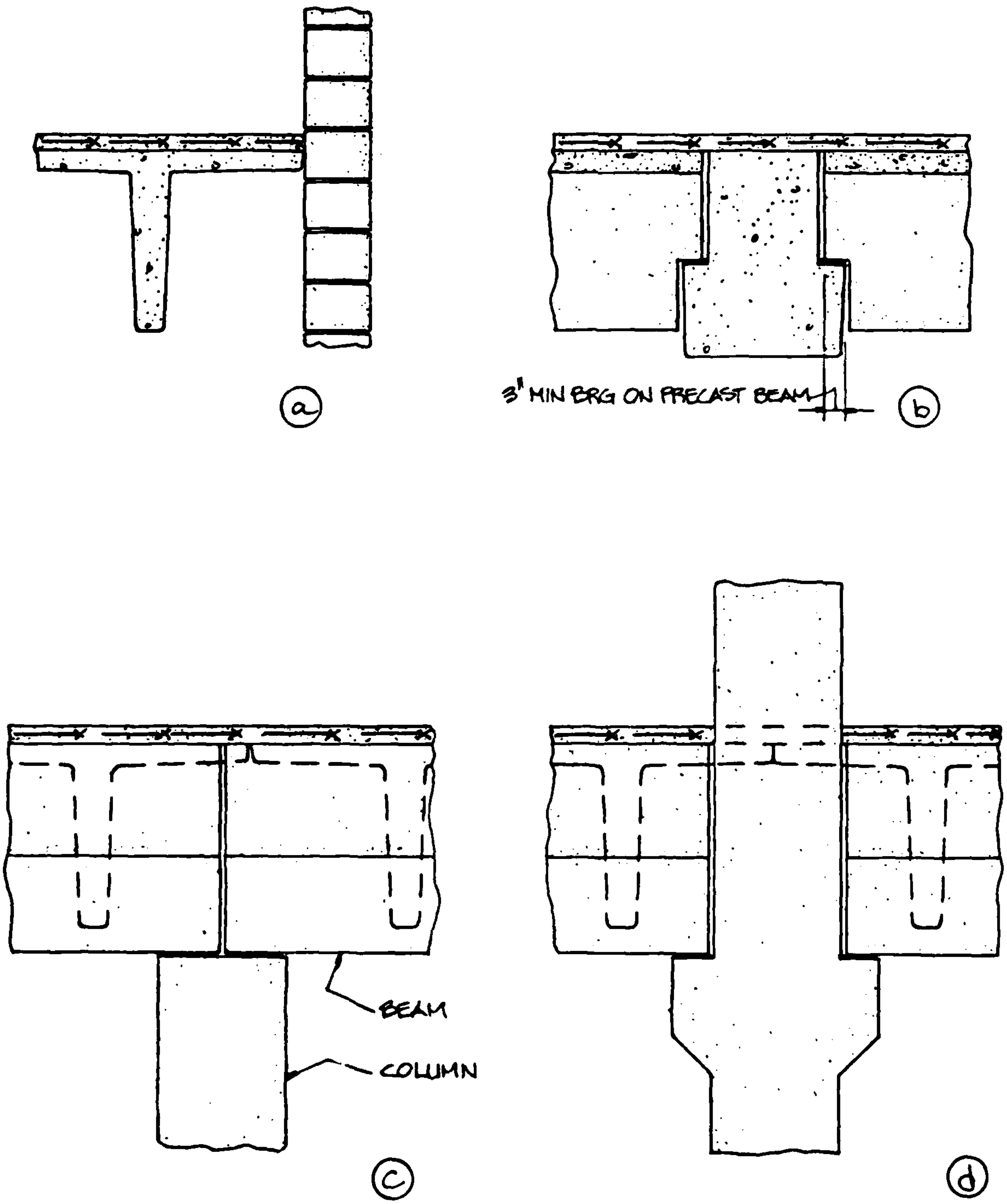


Fig. 4-14. Double and Single Tee Connections (Topped).

practice that has been generally discontinued. As might be expected, restraining these units to this degree against volumetric changes can result in considerable distress that, in several cases, has resulted in collapse. One of the more recent notable examples was in Antioch, California, where a partial collapse of a high school auditorium/gymnasium roof occurred after 21 years of service (Ref. 18). Although there were several contributing factors associated with the collapse, it was concluded that the added stresses resulting from volumetric changes triggered the event.

Another support configuration common in double and single tees is that of a notch, or "dap", at the ends of the members. This is where the connection is recessed, or dapped, into a member during casting, as illustrated in Figure 4-14(b). The purpose of this type of connection in stemmed members is to provide a level floor or roof above when members of different depths frame into one another, or to accommodate architectural requirements with respect to floor-to- ceiling heights or overall height of the structure. These types of connections present special problems to designers because of the several potential failure modes that must be investigated separately. It is generally accepted by the design profession that many of these notched connections were inadequately reinforced prior to 1970, a view that is supported by continuing evidence of distress in this area in older structures. In the last ten years, the design approach to this connection has been significantly revised, and structures constructed during this time probably contain adequate reinforcing. However, because of the special problems that must be addressed in its design and fabrication, these types of connections require investigation with respect to probable failure modes and possible upgrading techniques, prior to the consideration for use in candidate shelters.

All of the above floor and roof elements may be installed in a building with or without structural reinforced concrete topping. When topping is used, it is applied at the building site on top of the previously installed precast floor and/or roof elements. The topping serves several purposes in this type of construction. It may be used to assure a level floor surface, since prestressed precast units generally contain inherent camber, or upward bow, particularly if the spans are long, and to eliminate the problem of differential camber between adjacent units. It may be

used to increase the structural load-carrying capacity of the units by designing the topping to perform compositely with units. However, its primary use may be to serve as a lateral load carrying diaphragm in areas of the country where seismic design is a prime consideration. When topping is used, it is normally 2 to 4 in. thick.

If concrete topping is not used over the tee units, the differential camber problem is solved and the lateral diaphragm developed by the welding of embedded plates cast into the flange edges. Untopped solid and hollow-core slabs, depending on the design and fabrication method, use either welded edge connections, or achieve the leveling and transfer of the lateral forces by use of the grout keys between units. This type of construction is prevalent in the eastern and southern areas of the country.

It is obvious from the above that individual precast concrete roof and floor elements have very little positive resistance to large dynamic loadings in any direction except downward. Horizontal loads can only be resisted by bearing friction, which is negligible if the elements are properly installed with elastomeric or frictionless bearing pads, and upward loading is resisted only by gravity. When the units are tied together and the resulting diaphragm secured properly to shear walls, but untopped, additional resistance is achieved horizontally (particularly if the units are seismically designed) but little in the upward direction. The primary problem in the use of this material as untopped floor or roofs in buildings that are considered for use as shelters is the previously mentioned one of redundancy. None of the connection methods outlined above is strong enough to transfer large dynamic loading throughout the system. Individual units would be expected fail, and would, in all probability, drag adjacent units and parts of the supporting structure down with them. Evidence of this type of behavior was observed during a test of untopped hollow-core units subjected to 40 psi at the MILL RACE event (Ref. 8). Without considerable further testing and evaluation, along with the development of upgrading techniques, it would not be recommended that structures with untopped precast concrete floors or roofs be considered for use as shelters.

Although there are no test data for substantiation, it would be expected that precast floors and roofs topped with structural concrete would perform better. The topping is continuous over a number of the units and is connected to the vertical lateral resisting elements (usually shear walls). The topping would assist in holding the units together and in resisting movement away from supporting elements. It is possible that the use of some form of topping that could be expediently applied would serve to some degree as an upgrading method for untopped systems. Testing of these units with topping is required in order to develop the required survival parameters for these systems so they may be included as possible shelter options.

Beams and Girders

The differentiation between the term "beam" and "girder" is not a significant one with respect to this discussion. If any distinction is required, it is usually required only for clarification during discussion of a number of similar, but different size, structural members in a particular building. In those cases, the beams are the smaller members and may be supported by girders. The design, fabrication, erection, and performance of beams and girders are essentially similar, and in order to minimize confusion, we will use only the term girders to be consistent with the terminology of the precast concrete industry.

Precast concrete girders may be fabricated using either prestressing strands or conventional mild steel steel reinforcing as the primary reinforcement. Generally, because of the economics involved, the large majority of girders produced today are prestressed. This was not true 20 years ago. However, the methods of reinforcement have no bearing on an investigation of the connections, nor do they significantly affect the anticipated performance under a large, one-time, dynamic loading.

Girders are normally used in a structure to support other precast concrete floor or roof components, such as the solid and hollow-core slabs and the tee units described above. Their purpose is to transfer the vertical loads from these components to the columns and/or wall systems. Although they may be manufactured in many cross-sectional shapes, they are usually classified in three general categories: rectangular, ledger, and inverted tee. They are normally fabricated in

depths up to 48 in. for use in buildings; larger sizes, however, as well as additional shapes ("I" shapes, etc.) are not uncommon in bridge construction. The standard cross sections are shown in Figure 4-15.

The end bearing details and connections for girders are basically the same as those used for tee units, as described previously. Some of the more typical are shown on Figure 4-16. They may be bearing on elastomeric or other types of bearing pads, or may have a welded connection at one end, and frequently have notched, or dapped, ends, as shown on Figure 4-16(c). The connection shown in Figure 4-16(d) consists of reinforcing bars projecting into tubes cast into the ends of the girder and is not designed to provide significant restraint. Precast girders are rarely designed with continuity at intermediate supports, but when the precast floor elements are topped with structural concrete, they are usually designed compositely with the topping.

Since precast concrete girders have many of the same connection characteristics and are designed similarly to tee sections, it would be expected that their performance under blast loading would also be quite similar. This is true to some degree. Girders supporting floor units that have structural topping will certainly perform better than those supporting untopped units. However, unlike single or double tees, girders have much more stability and mass, and do not rely on the relatively thin top flanges for integrity. For example, single tees are extremely unstable unless lateral support is provided by adjacent units or walls, a fact that is taken into account during their erection. Girders, on the other hand, are relatively stable whether supporting a floor or not, and their mass alone makes them resistant to being displaced by much lighter precast floor or roof units. Two precast girders were tested at the MILL RACE event (Ref. 8). Each of these girders carried untopped hollow-core slabs on either side and was supported at its ends on concrete corbels without any positive connection. Both girders were shored to partially withstand the 40 psi overpressure. Although the slabs on either side failed almost completely, the girders remained in their original location with only slight damage.

Areas with respect to precast girders requiring investigation include the notched end bearing configuration, and the development of upgrading techniques to

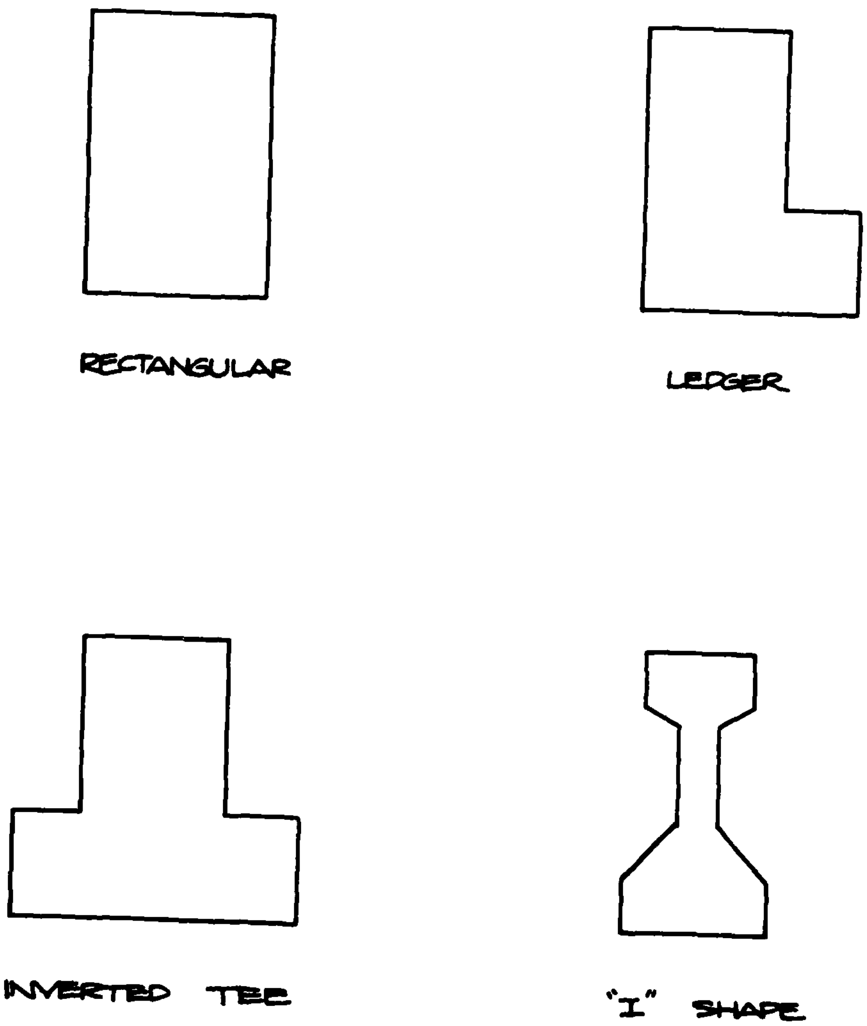


Fig. 4-15. Girder Shapes.

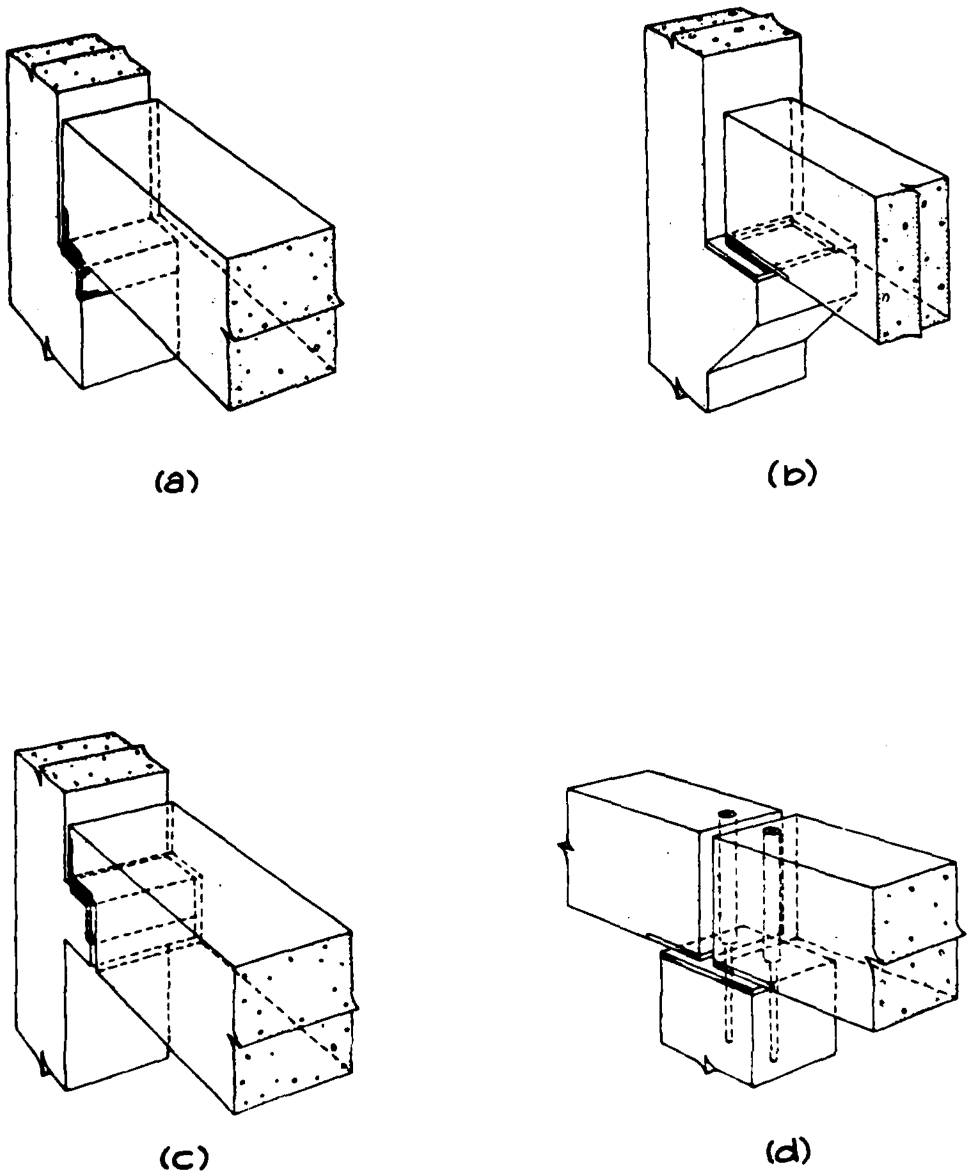


Fig. 4-16. Girder Connections (Ref. 17).

resist horizontal translation. Existing hardware, with some modification, now used on bridge girders to resist lateral and vertical motion as a result of earthquakes, may hold promise in this latter area as an upgrading method.

Columns

The performance under blast loading of precast concrete columns and their related connections cannot be discussed in the same context as other precast concrete elements for several reasons. First, from a practical standpoint, precast concrete columns cannot be upgraded, nor can their primary connections, at the base or column to column, be strengthened or upgraded to any degree. Second, the ability of a precast column to perform as designed; i.e., to remain vertical and support the precast elements resting on or connected to it, is to a large degree dependent on the performance of these elements and their connections. The state of the art of the use of precast columns is such that these elements are normally only used as vertical load resisting elements in buildings (Ref. 15) and, by themselves, have little resistance to lateral loading. In a completed structure, the precast floor elements, tees or slabs and girders, provide some degree of lateral support for the columns, but if these elements partially or wholly collapse, the columns would not be expected to survive.

Precast concrete columns may be made in any cross section, but square or rectangular cross sections are the most frequent. They normally range in size from 10 in. by 10 in. to 30 in. by 30 in., and seldom are longer than about 50 ft (5 stories). If the columns are longer than one story in height, corbels will be located on one or more sides in order to provide support for the precast girders. Precast columns may be either prestressed or conventionally reinforced with mild steel, with the use of mild steel greatly predominating.

The typical base connection for precast concrete columns is achieved by use of a bolted base plate (Ref. 16). This connection consists of a steel bearing plate, with predrilled holes, cast on the bottom of the column and welded to the vertical main reinforcing steel of the column. Anchor bolts are cast in the footing, and during erection the column is positioned over the bolts and set to the desired vertical alignment and elevation by the use of leveling nuts on the bolts. Figure 4-17 shows

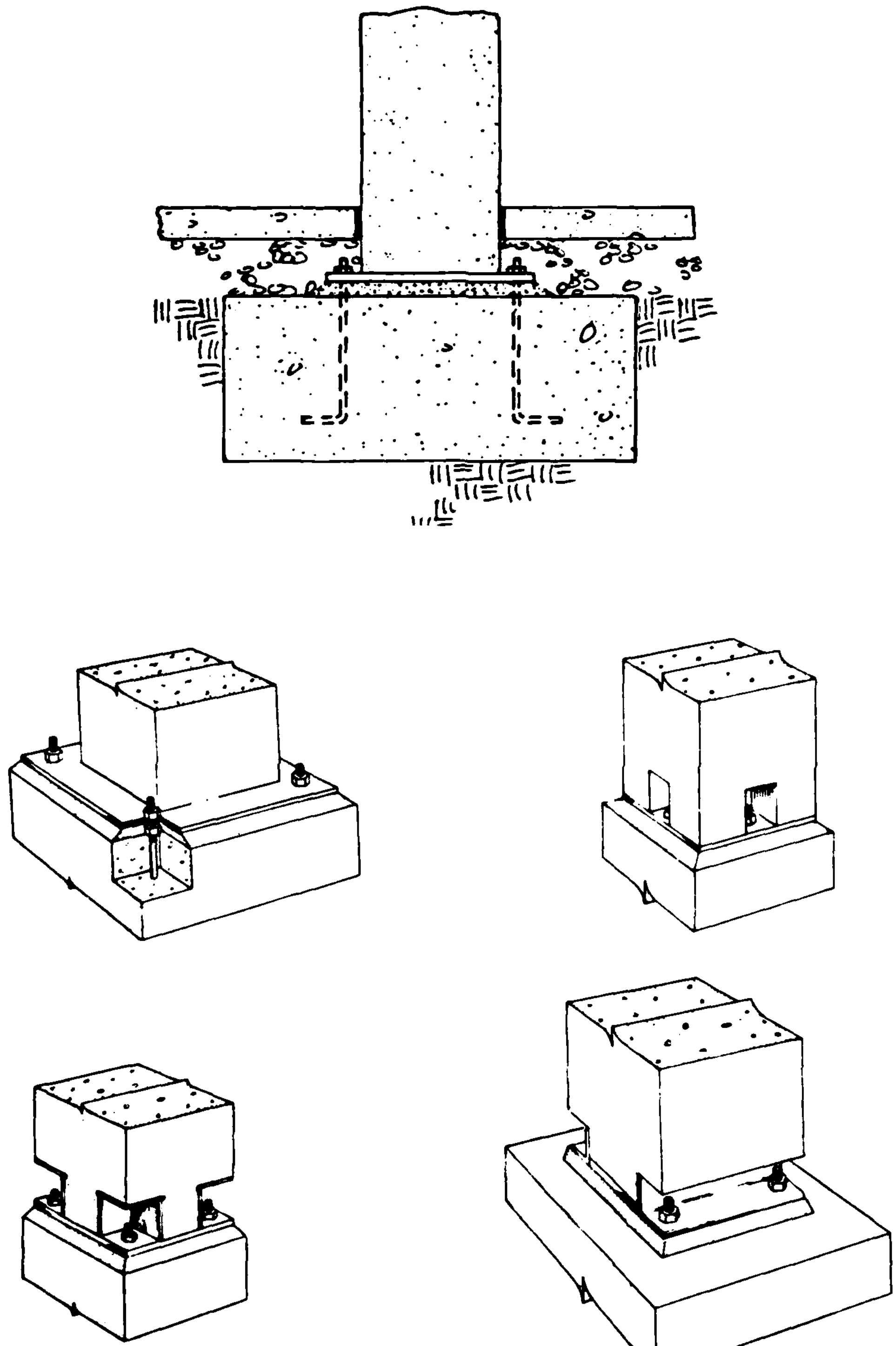


Fig. 4-17. Precast Column Base Connections (Ref. 17).

several different configurations of these connections. The column is braced in this position, and the space between the bottom of the plate and the top of the footing is drypacked, usually with a non-shrink, high strength grout. The design of this base connection is based on both the erection loads and the vertical loads that occur in service, and, as mentioned above, the connection has little ability to resist lateral loads. This connection design is an important consideration in the performance of this type of building when subjected to lateral loading, particularly in relatively stiff shear wall buildings (Ref. 15). The proper design and construction of bolted base plate connections, along with the use of the proper elastomeric bearing pads at the beam/corbel interfaces, allow sufficient drift movements to be accommodated in the total structure without distress.

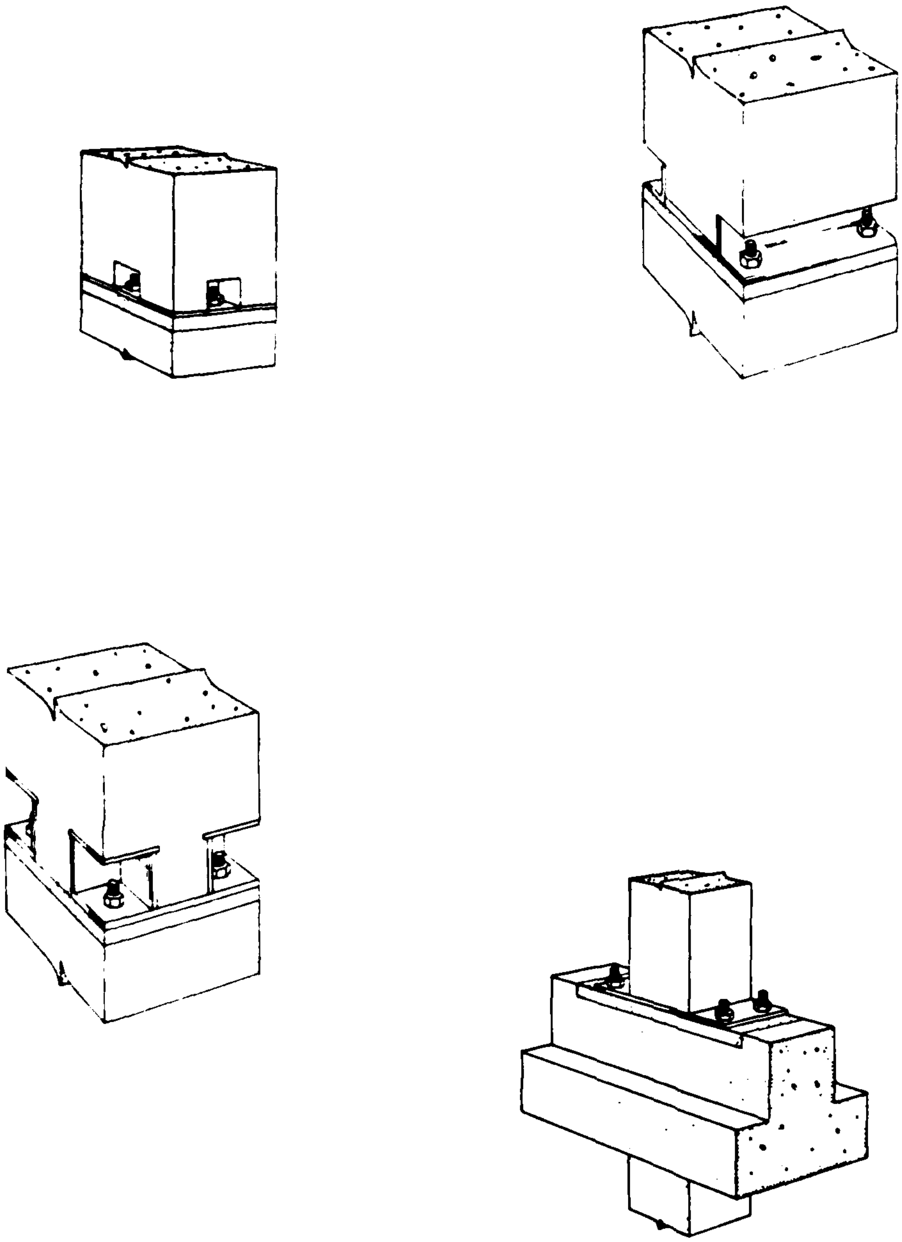
If the columns require splicing together, either to achieve a longer column or to provide for some type of discontinuity, the connections are similar to the base connections, and are shown in Figure 4-18.

The primary difference between cast-in-place and precast columns in performance under unusual loadings, particularly if applied laterally, is significant. A cast-in-place column is cast around reinforcing steel protruding from the footing. This steel is lapped with the main column steel, and even if not designed as a moment resisting connection, it provides considerably more structural redundancy than the bolted plate connection by distributing unwanted stresses to the footing. Cast-in-place columns in otherwise all precast buildings are not unusual, and, in general, they would be expected to have superior performance when compared with precast columns under blast loading. Figure 4-19 shows two typical cast-in-place column connections.

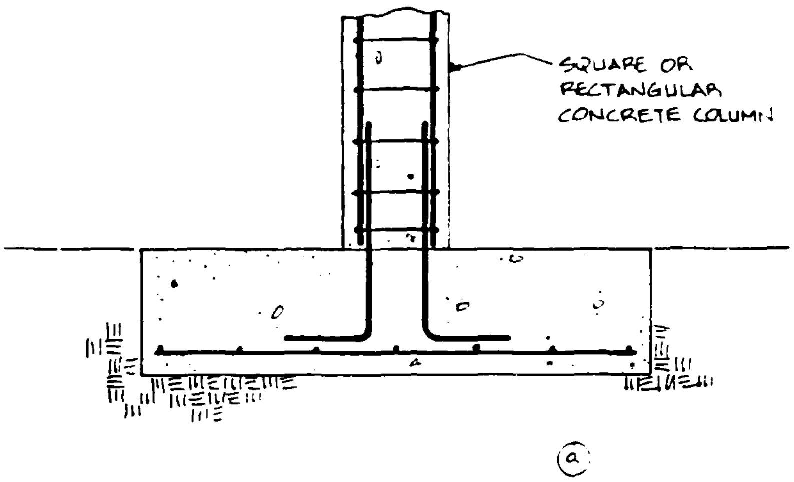
Wall Panels

Precast concrete wall panels are modular elements used to form the envelope of the building. These wall panels fall into three basic categories (Ref. 15):

(1) Non-load bearing cladding panels - designed to support only their own weight and wind or seismic forces normal to the panel - may be spandrel panels, solid wall panels, or window panels.



. 4-18. Precast Column-to-Column Connections (Ref. 17).



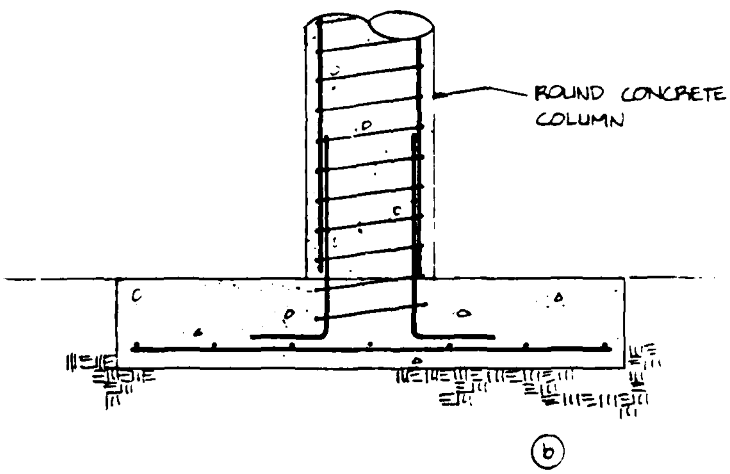


Fig. 4-19. **Reinforced Concrete Cast-in-Place Column Base Connections.**

(2) Non-load bearing shear walls - designed to transfer lateral wind or seismic forces from the horizontal diaphragm to the foundation or other elements.

(3) Load bearing wall panels - designed to support vertical loads from the building framing system, and may also be designed to transmit lateral forces to the building foundation.

Cladding panels, the first basic category, are generally architectural and do not contribute structurally to the integrity of the building. These types of panels should be recognized as a possible hazard in structures selected as shelters; as mentioned previously, however, this performance aspect is not under consideration in this section of the report.

Non-load bearing shear walls, the second category, are normally one-story high walls, usually interior. Since they are not designed to carry vertical loads, they would, in most cases, be found running parallel to precast elements used in the floor or roof, such as tees and slabs. It would be impractical in most cases to use these walls perpendicular to the precast elements, since their function requires a shear connection between the top of the wall and the horizontal diaphragm, probably requiring penetration through the bottom of the floor or roof elements, usually a difficult and costly task. Also, if in the building layout, this wall was required to run perpendicular to the precast elements, it would be more practical to shorten the floor elements, bear on the wall, and design the wall as a load-bearing shear wall. These walls, because of their intended purpose in the overall structural scheme, are normally securely connected not only to the horizontal diaphragm, but to the foundation. This is particularly true in areas of high seismic risk. Since these walls are designed to resist lateral loads and would be instrumental in providing stability to buildings subjected to blast loading, and would also probably assist in carrying some of the vertical loads if required to do so, they would be considered as helpful additions to the main structural system.

The third category of wall panels, load bearing wall panels, is the most important type with respect to maintaining the structural integrity of a precast concrete building. These walls are not only required to support the floor or roof

elements, girders, slabs, or tees, but in most cases they are designed to transmit lateral forces to the foundation, or to act as shear walls. Because of their significant contribution to the performance of a precast building, these types of precast wall panels and their effect on the use of these types of buildings as shelters will be discussed in more detail later in this section.

In general, all three of the panel categories have much in common. They may be plant cast, hauled to the building site, and erected in final position, or may be cast at the site, adjacent to their final position, and "tilted up" into position. They may be prestressed (if plant fabricated) or use mild steel reinforcing. Wall panels may be fabricated in any size desired, consistent with the economics of the hauling and erection equipment, but generally, 20 ft to 24 ft in width and up to 60 ft in height are the practical maximum, with the larger of these sizes usually cast at the site. Coloring admixtures are sometimes added to the concrete mix, and exterior finishes are available in a wide range of textures from smooth to deeply exposed aggregate. These panels are also sometimes cast as sandwich panels, where a center core of some type of insulative material is sandwiched between two layers of concrete, thus making them more energy efficient.

Many times, the precast sections previously described as floor and roof elements are used as wall panels. The most common sections used for this purpose are the double tee and hollow-core sections. Some hollow-core sections, owing to their manufacturing processes, are not suited for use as wall panels in areas where high wind and seismic loading are a consideration. The reason for this is that the methods of fabricating these sections do not permit the proper embedment of the panel-to-panel connections required for these high magnitudes of lateral loading.

Load-bearing wall panels may be up to three stories in height, may be basement walls (and therefore act as retaining walls), may be installed on the interior or exterior of a building, and may be designed as lateral load resisting elements. Each of these uses, as well as the lateral load requirements as specified by the local building codes, influences the type, size, and number of connections required.

Connections for load-bearing wall panels are subject to heavier loads and more varied loading and load transfer conditions than connections of non-load- bearing wall panels. These connections become an essential part of the structural support system, and the stability of much of the building depends upon them. In addition to the weight of the panels, the connection may be required to resist and transfer dead, live, wind, and earthquake loads, as well as the effects of volume changes, and the panels may have both horizontal and/or vertical joints across which these forces must be transferred (Ref. 19.)

The distribution of lateral forces to a bearing wall depends largely on the diaphragm action of the floor and/or roof systems, and therefore adequate connections of these systems to the wall are essential. Additionally, the force transfer mechanism from the wall to the support structure, or foundation, must be capable of transferring not only all these lateral forces, but the vertical forces as well. Since these connections, which normally occur along the bottom and near or at the top of the wall panel, are most critical, they usually receive a significant amount of attention during the design and construction phases of a project. Again, this is particularly true in areas of high seismic risk. Following is an outline of some of the types of connections that might be expected at the floor and/or roof intersection with the wall panel, and at the interface of the wall with the foundation.

Wall to Floor or Roof Connections - Two types of distinct functions may occur and must be provided for with respect to these connections. One is that some type of support must be provided at the bearing ends of the precast floor or roof elements, whether solid or hollow-core slabs or tees; and second, the diaphragm, developed either in the precast element itself or in a structural concrete topping placed over the elements, must be connected to the wall. A number of typical wall to floor and roof connections are shown on Figure 4-20.

End support for the precast floor or roof units by precast wall panels is usually accomplished by setting the units on continuous corbels cast into the walls, on steel angles secured by inserts to the walls after their erection, or directly on top of the

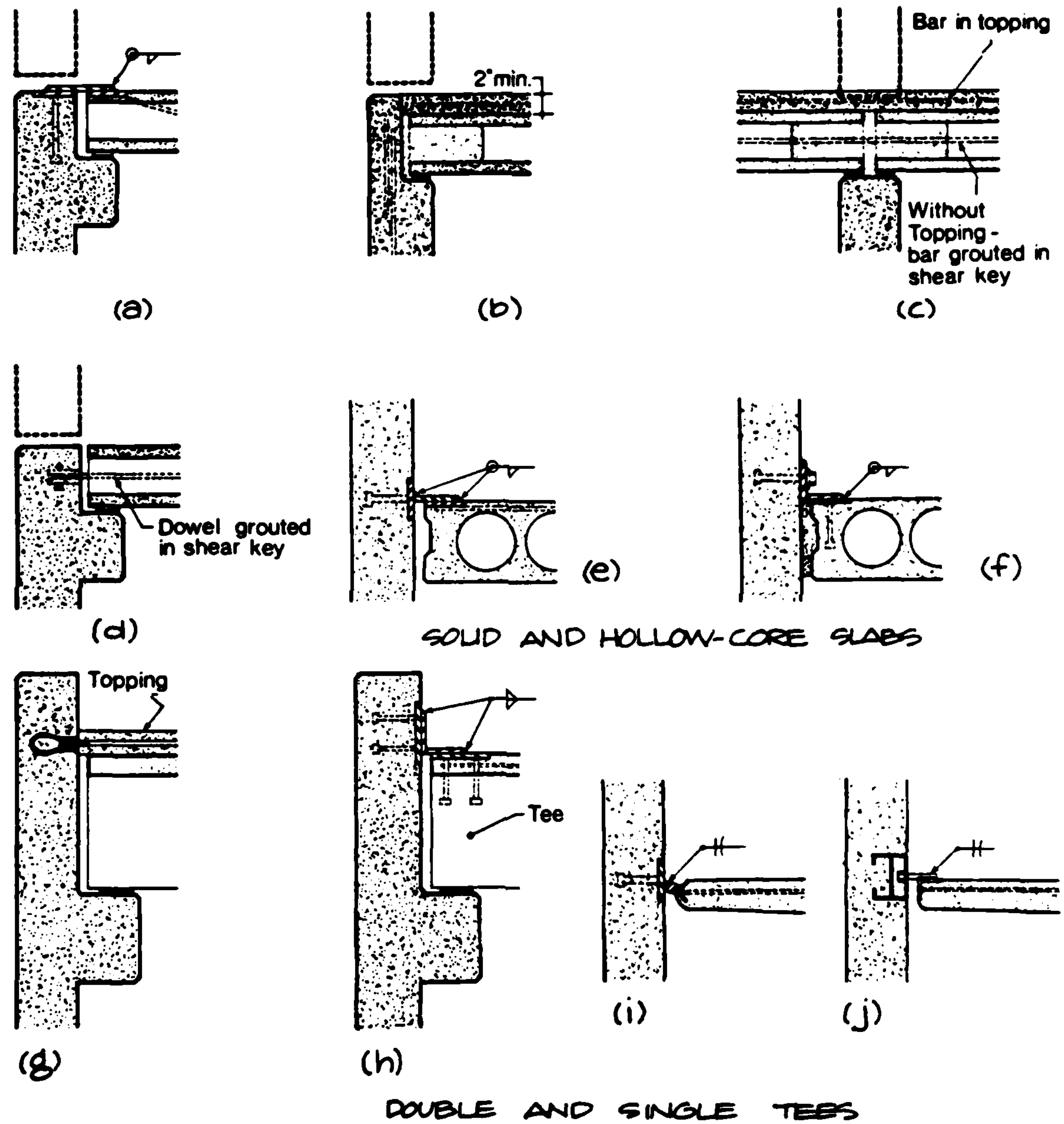


Fig. 4-20. Wall to Floor or Roof Connections (Ref. 19).

walls, which is usually the case with an interior wall. See Figure 4-20(a)(b)(c)(g)(h). Previously in this section of the report, a detailed description of the various types of bearing connections used for precast concrete floor and roof units was presented. These connections, and the reasons they are used, are also applicable to bearing of the units on wall panels. Solid and hollow-core slabs normally are supported on bearing pads and rarely have a positive end connection. Single and double tees may be supported on bearing pads, or may be welded at one end, achieving a "semi-fixed" bearing condition at that end. No bearing support is provided at the walls parallel to long direction of these units.

If the precast walls are also designed as lateral load resisting elements, connections must be developed between the floor or roof diaphragm and the walls. These connections would normally be required along all interfaces of the diaphragm with the walls, not just at the bearing ends of the precast floor units. See Figure 4-20(e)(f)(i)(j). When the precast units are untopped, these connections are usually made by the welding of inserts cast in the floor units to inserts cast in the wall, or, sometimes in the case of solid or hollow-core slabs, by connecting reinforcing steel located in the grout keys, or joints, between slabs to inserts cast into the wall, either by threading the reinforcing bar into the insert or by welding. If the floor units are topped with structural concrete, and this topping is to perform the function of a diaphragm, connections to the wall may be accomplished by casting reinforcing steel dowels in the wall, protruding into the slab area where the topping is to be placed, or by threading these dowels into inserts previously cast in the wall, Figure 4-20(g).

Wall to Foundation Connections - These connections may be quite massive or relatively light, depending on the load requirements. They may have to accommodate vertical uplift forces as well as the lateral, or base shear, forces. There may be numerous light connections, spaced close together along the wall/foundation interface, or there may be several significant connections per wall panel. In selecting the connections, the designer must make provision for leveling and aligning the panel, access to the connection by the required trades, and development of the necessary force transfer mechanism. When a few connections are made per panel, they are usually accomplished by large steel plates, welded to

the reinforcing steel in the wall panel, cast into the wall, and welded or bolted to plates cast into the foundation. If this method is used, blockouts, or access cavities, are usually cast in the wall at the location of the plates for worker's access, and then grouted in later. If many lighter connections are to be used, these are generally made by small angles or plates, welded or bolted periodically to the embedded wall and foundation plates. Other connection methods sometimes employed include grouting protuding wall dowels into recesses provided in the foundation, and splice sleeves for connecting the wall steel to the foundation steel. Several of the most typical of these connections are shown in Figure 4-21.

Panel-to-Panel Connections - To assure lateral force transfer between adjacent walls, the vertical joints between may be required to transfer either direct tension and compression or a combination of tension and compression with vertical shear. Horizontal joints may be subjected to the effects of lateral and vertical loads, in compression and possibly tension, and horizontal shear. Vertical and horizontal joints, formed by two or more panels, may be connected to form one structural unit or to act independently. Accordingly, the types, sizes and number of panel-to-panel connections vary widely and are difficult to categorize.

Vertical joints may have grooved or keyed joint connections, which may be reinforced or unreinforced. They may have mechanical connections consisting of anchorage devices cast into the wall panel and steel sections (plates, angles, bars, etc.) crossing the joint at locations between the top and bottom of the wall. Figure 4-22 shows many of the typical vertical panel-to-panel connections.

Horizontal joints in load-bearing wall construction occur usually at floor levels and at the transition to the foundation. The wall to foundation connections, which were presented previously, are a special case of horizontal joints. The wall to floor or roof connections discussed above may occur at a horizontal panel-to-panel joint, particularly if it is an interior building panel with precast floor units bearing on it from both sides. Depending on the type of floor units and their support details, three kinds of joints can be distinguished: thin grout joints, open joints, and closed joints (Ref. 19), as shown in Figure 4-23.

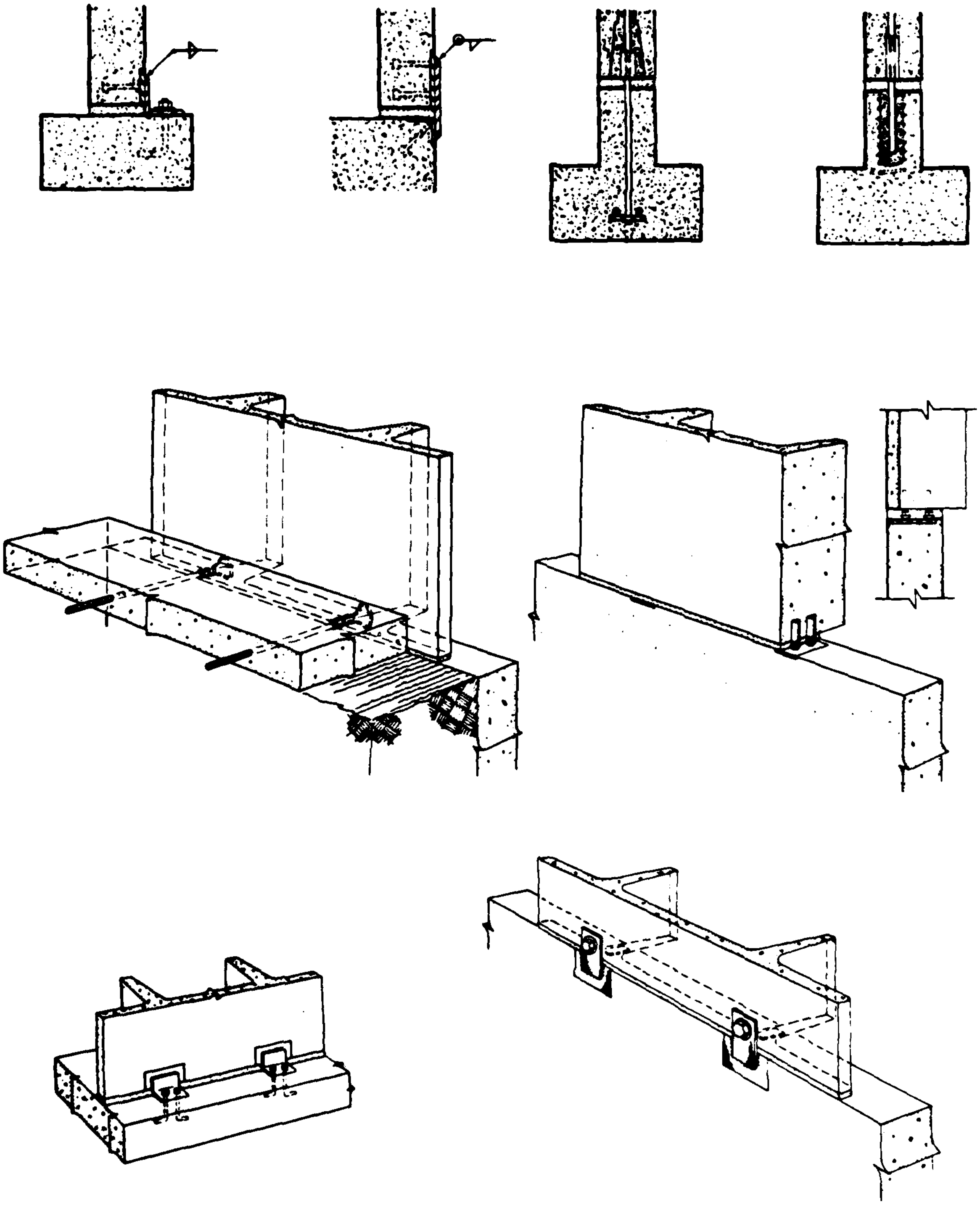


Fig. 4-21. Wall to Foundation Connections (Refs. 17 and 19).

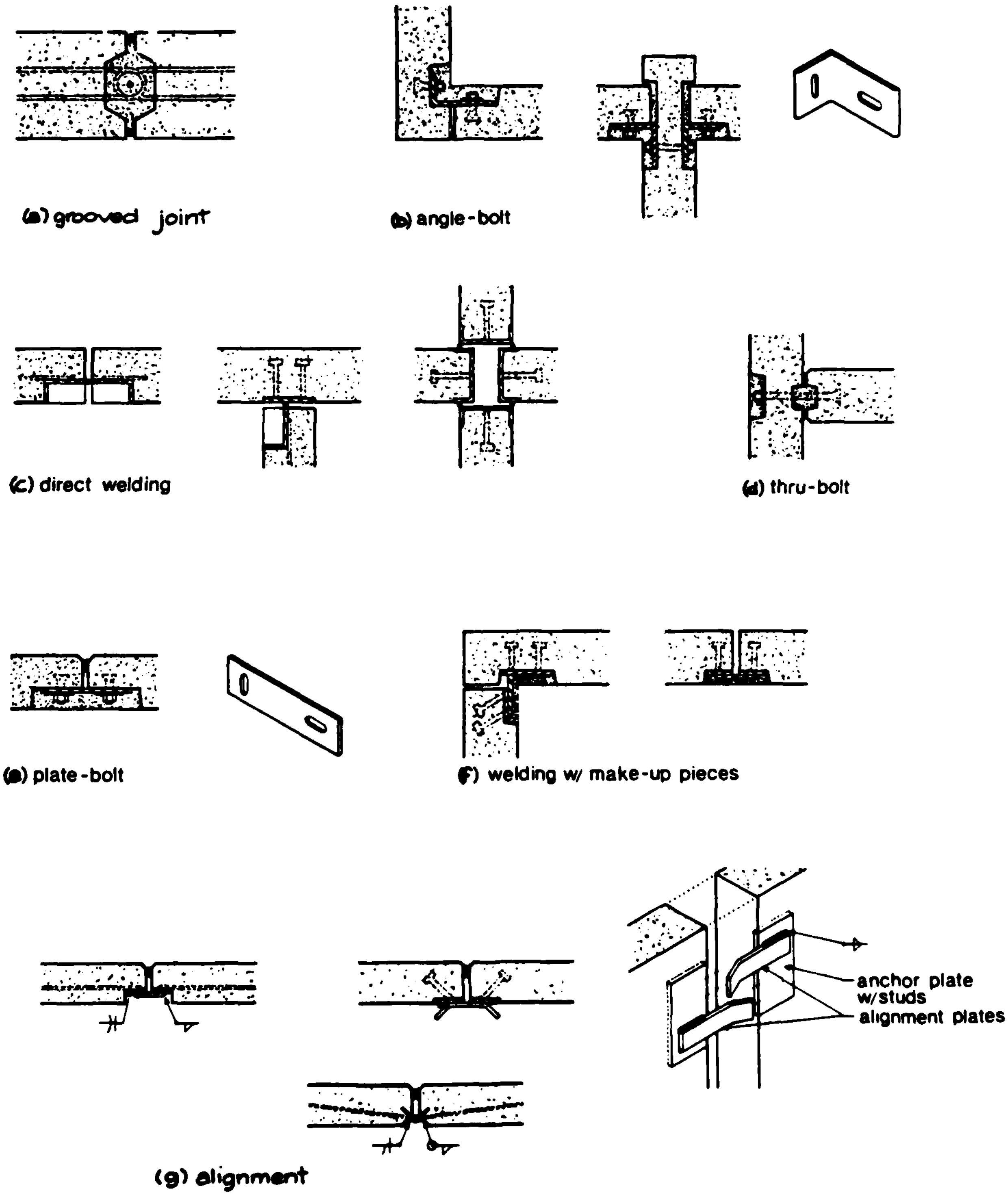


Fig. 4-22. Vertical Panel-to-Panel Connections.(Ref. 19).

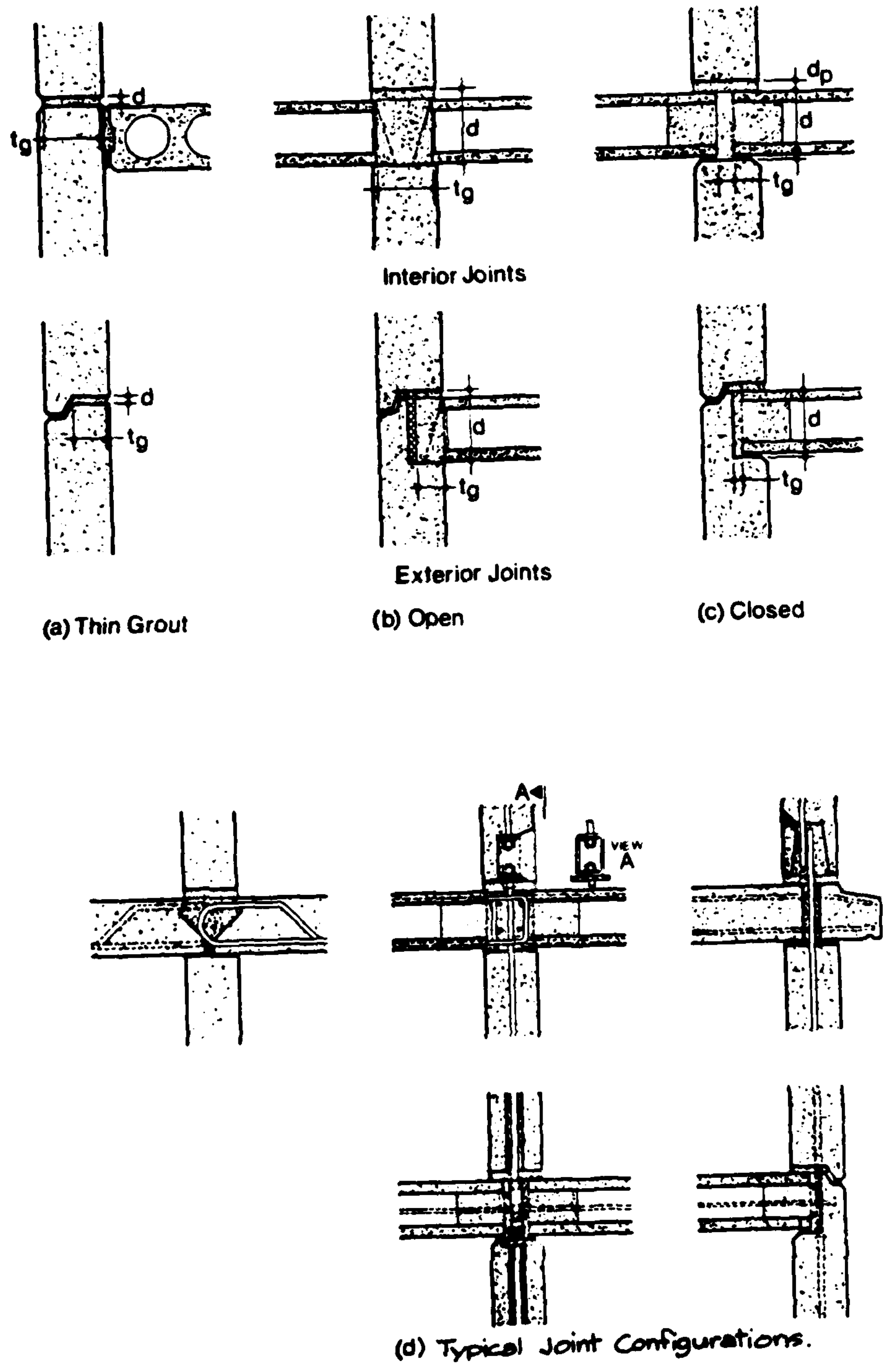


Fig. 4-23. Horizontal Panel-to-Panel Joints (Ref. 19).

Thin grout joints are mainly used for shear walls and for bearing wall tee units. In these joints, the wall panels are located one on top of another with a thin grout joint between, and the strength of the joint is basically controlled by the wall strength, see Figure 4-23(a). When tension or horizontal shear forces are present, these forces are transferred by the use of mechanical connections or reinforcing steel. These types of joints are independent of the location of the floor to wall connection.

Open joints are used with solid or hollow-core slabs when exterior and interior floor to wall connections occur at the horizontal panel joint, but where the slabs do not penetrate over the top of the wall below, see Figure 4-23(b). The vertical width of the joint is usually the thickness of slab, and its strength is dependent on the strength of the grout, drypacking, or concrete used to fill the joint. Since this joint is also a floor-to-wall connection and would be required to transfer the lateral diaphragm loading from the floor to the wall, the methodology for this type of connection, as previously discussed, is applicable; i.e., mechanical or reinforcing steel connectors would be required.

Closed joints are also normally used with solid or hollow-core slabs when the interior or exterior floor to wall connections occur at a horizontal panel joint. This is the most common type of joint used in precast slab/wall panel systems. In these joints, the slabs extend over top of the wall below, from one or both sides, such that the panel above is partially supported by the floor slabs, see Figure 4-23(c). In order to develop shear resistance to lateral forces, connections of reinforced concrete, plain concrete with vertical ties, or bolted or welded connections are used. Tensile stresses at the joints are usually handled by vertical ties in the form of welded reinforcing, overlapping loops, grouted dowels, or angle bolt connections. Typical reinforced joint configurations are shown on Figure 4-23(d).

Precast concrete wall panels, when taken as an independent structural component, would be expected to perform as well as cast-in-place walls. They would certainly be equal to or better than a reinforced masonry wall of equal thickness, and would be superior to unreinforced masonry or brick or block veneer. However, when these panels are examined in the context of a building system, as was

true with other precast components, the maintenance of the integrity of the wall panel connections during blast loading is the paramount factor relative to their performance. Generally, the connections between the wall and the foundation are substantial, but only in zones of high seismicity are the floor to wall connections significant. The panel-to-panel connections have little resistance to moment, or bending, perpendicular to the joint, and would not normally be expected to perform well under lateral dynamic loading. If the horizontal panel-to-panel connection should occur at the same location as the floor or roof to wall panel connection, additional lateral resistance might be anticipated.

Areas that require consideration for further investigation are the horizontal panel joint connections, particularly those that indicate the most promise of integrity and possible upgrading; i.e., at the foundation and at the floor or roof level. Two precast wall panels were tested at the MILL RACE event as basement walls at a 40 psi overpressure, but were specifically designed so that the connections were not a factor in evaluating the test results (Ref. 8). Since precast wall panels are increasing in use throughout the country, tests are required on these wall panels with realistic connection conditions, particularly at the floor or roof level in a basement structure at greater than 40 psi, and at the one story and the foundation level in aboveground buildings at something less than 5 psi.

Commentary

As one reviews the above discussion on precast concrete elements, connections, and the resulting building systems, several facts become apparent: (1) Although the individual structural precast components are finite in number and can be generally grouped together for evaluation, the methods and hardware used for attaching and securing these components together vary greatly and approach infinity. (2) A building constructed primarily of precast concrete components has little redundancy, and therefore may be unable to redistribute unwanted stresses caused by overloading adequately throughout the structure. (3) A precast concrete structure may only perform as well as the performance of the minimum components and/or connections, and isolating the collapse of one or more of these components might not be possible, thus resulting in progressive collapse of a significant portion of the structure.

The above facts appear to severely discredit precast concrete construction for consideration as a shelter. However, the economics of this type construction are such that a large number of these buildings is now in place, and the industry is expected to grow substantially in the future. Accordingly, this type of construction cannot be excluded from the overall shelter program. Ways must be found to evaluate these structures and their component connections, and to upgrade, if possible, at least the critical connections and/or portions of the building for use as shelters. Two current FEMA/SSI programs -- studying the demolition of existing buildings and the response of building frames to dynamic lateral loading -- may yield some useful data in this regard.

SUMMARY

The expected performance of cast-in-place concrete floor and roof slabs subjected to blast loading, with several exceptions, is generally good. Two-way slabs and flat slabs, in the shored condition, have been subjected to 40 psi overpressure in field tests with good results (Ref. 8). A number of static tests have been conducted on one-way slabs, both shored and unshored (Refs. 2 and 9), as well as dynamic tests (Ref. 10), all indicating satisfactory performance. Waffle slabs subjected to dynamic loading have performed well, as have one-way joists. Tests, as well as analysis, indicate somewhat less performance might be expected from a flat plate system, but the deficiencies appear to be correctable with proper upgrading techniques. Post-tensioned slabs appear to be questionable with regard to their performance under blast loading, and additional research is required to determine if any upgrading scheme is practical for these systems.

When evaluating cast-in-place concrete buildings as a total system, one would expect that, with the proper upgrading, their performance would be superior to the majority of building systems. The method of construction, the casting of the floor or roof, columns, and sometimes the walls, monolithically, developing to some degree moment-resisting connections, gives these systems the distinct advantage of redundancy, and thereby the ability to redistribute stresses resulting from overloads throughout the structure. Unfortunately, precast concrete structures lack this

ancy. In general terms, their overall performance as shelters would be t. Tests on hollow-core floor slabs have been conducted both statically (Ref. l in the field at 2 and 40 psi overpressures (Ref. 8), with mixed results. e of the large number of these types of systems in existence, as well as the ing use of this method of construction, a considerable amount of investigative is in order to determine possible upgrading techniques and methods of ing, or improving, connections.

)ne approach that holds promise in the evaluation and subsequent upgrading of gs is that of attempting to tie the lateral design criteria to the anticipated g performance. One common theme that was prevalent in the preceding ion on concrete construction was that the integrity of the connections, and ore the building system, was tied more or less directly to the design dology. If a building is designed to withstand significant lateral loading, because of code requirements or for other reasons, its performance as a would be expected to be superior to one that is designed and constructed t such requirements. This would be true with all types of structures, but te structures, and particularly those of precast design and construction, are ly sensitive to lateral load design requirements.

Cast-in-place concrete buildings designed to accommodate large lateral loading e ductile moment-resisting frames in which the connections are specifically ed to prevent failure in the joints and confine any yielding to the flexural ers (girders) rather than the columns, thus, with the exception of unbonded ensioned systems, providing additional resistance to possible catastrophic es. Precast concrete buildings normally use shear walls as lateral load resist-ements, and large lateral load requirements result in substantial diaphragm tions between the floor or roof and the shear walls, between the walls and the tion, and even in the non-structural architectural element connections. These tions, depending on the lateral load requirements, vary considerably hout the country, and run from virtually nonexistent in many areas to quite ntial in others. Although precast concrete buildings may be judged to be far on the list of shelter options, the ability to evaluate these structures as to elative performance is important to the overall shelter program.

Lateral load requirements for buildings are based on the anticipated horizontal loading resulting from wind or earthquakes. Basically, therefore, the question of defining these requirements is, to a great degree, based on the geographical location of the building. It would appear then that by using published maps, such as those in Chapter 23 of the Uniform Building Code (Ref. 5), indicating the various zones of wind and seismic activity, one would have a simple guide to the design parameters used in particular areas. Unfortunately, however, the solution to the problem is not that straightforward. The UBC is only one of a number of building codes used in this country, and each may have slightly different risk zone maps, and more important, may require different design requirements for an identical risk zone. The reasons for these design discrepancies are primarily psychological and not technical. Portions of the country that have experienced severe wind or earthquake damage, particularly in recent history, are much more conscious of the effects of these events, and emphasize their design philosophy accordingly.

For example, the Uniform Building Code is one of the better building codes with respect to lateral force design. It was developed initially in California, an area of high seismic risk and with a history of recent catastrophic earthquakes. This code is widely used, with only minor variations, throughout California as well as neighboring states. In recent years, this code has been adopted for use in some midwestern and eastern states and cities, but it is significant to note that even in identical seismic zones (zones of supposedly equal risk) the design methodology used for buildings may be considerably different. Sacramento, California, and Charleston, South Carolina, for example, are both located in seismic risk zone 3 according to this code, a zone that might expect "major damage" from an earthquake. However, probably because of its proximity to San Francisco, both geographically and politically, buildings in Sacramento are designed with more attention to and emphasis on lateral force design, than those in Charleston. Each city has a different building code, which specifies a different approach to lateral force design, even in the same seismic risk zone.

Even with all the complexities involving different codes and design requirements, it is believed that connecting lateral load requirements to expected building performance is a viable approach, and it will be considered in the

development of a prediction methodology for the evaluation of shelter options. When viewing this concept on an overall national basis, there is obviously considerable diversity between geographical regions and political jurisdictions; this is not, however, a technical problem with respect to the shelter evaluation process. Only about a half dozen building codes are used throughout the United States, these are updated periodically, and records are available in all cities, counties, states, as to which of them was in use at the particular time. For example, there were no requirements for consideration of lateral forces as a result of earthquakes in the building codes used in southern California prior to 1933 and in northern California prior to 1948. Accordingly, buildings built prior to those years in those particular areas would require downgrading when evaluated as shelters, with these evaluations becoming progressively more positive as the codes in these areas recognized the seismic risk and reflected it in their design procedures.

Another area that will be incorporated into the evaluation of structures for use as shelters, and that ties directly into the lateral force design parameters, is that of the "importance factor" assigned to buildings. In several of the current building codes, the Uniform Building Code for example, both wind and seismic design take this factor into account. The factor is based on how essential the facility is in performing services during an emergency. Structures such as hospitals, fire and police stations, disaster operation centers, and buildings where the primary occupancy for assembly is large, say greater than 300 persons, are included. Although there is not a large number of these structures and this factor may not be significant in the overall number of buildings it affects, these types of buildings are usually large and well-built and should be judged accordingly and included in the prediction methodology.

PROGRAM PROJECTION

This section of the report covers the first phase of the investigation directed toward surveying and evaluating various types of structural connections, estimating their survivability, and establishing priorities for the testing and analysis that will be accomplished in the latter phases of the program. The current effort was directed

at connections occurring in concrete construction, both cast-in-place and precast, and primarily consisted of an extensive literature search and review of existing reports, design manuals, industry catalogs, and building codes, supplemented by participation by SSI personnel in building failure analysis seminars. The objective of these efforts was to determine past practices and performance history, as well as the current state of the art, in the design, testing, and construction of these types of connections.

It is projected that the program for next year will include the following:

1. Test and analyze various types of concrete connections based on the investigation conducted this year and reported in this section. Those to be tested include dapped end bearing connections and floor or roof to wall connections.

2. Survey and evaluate additional types of structural connections, primarily those used in steel and timber construction, for the purpose of estimating their survivability and establishing a required test program. It is anticipated that current investigations in the areas of building frame response and building demolition will add significant input to the survey and evaluation of structural steel connections.

3. Test and analyze several of the timber connections, particularly those used in glulam construction. This year's program included a number of full-scale static tests on glulam construction, and accordingly, it is possible to conduct the testing phase on these types of connections concurrently with the overall survey and evaluation of timber connections.

4. Begin to assimilate and format the data developed in this ongoing program on structural connection research so that it may be included in the prediction index portion of the prediction methodology now under development for use in the shelter upgrading manuals.

The following years of this program will be directed toward developing upgrading schemes for connections, as the requirements present themselves, and to additional evaluations and testing on wall connection systems.

Section 5
PUNCHING STRENGTH OF REINFORCED CONCRETE ROOF AND FLOOR SLABS

INTRODUCTION

The purpose of this investigation was to provide a method of evaluating the shore punching shear capacity of continuous slab systems, and to establish whether this can be a significant factor in the design of shoring systems. The investigation included the shore punching shear strength of both floor or roof slabs, such as flat plates, flat slabs, and two-way slabs, and floor slabs on grade, when upgraded with a given shore support configuration, were included in the investigation. See Figure 5-1a.

For each bay of a slab system, the shoring configuration typically consists of four or more shores in a symmetrical pattern, as shown in Figure 5-1b. The slab on grade is supported by soil with varying stiffness characteristics. From a review of past research on reinforced concrete slab behavior, some of the more critical modes of failure of shored slabs may be slab-punching, shore compression failure, flexural yield collapse, and slab-to-frame connection failure.

ENHANCED PUNCHING SHEAR STRENGTH IN CONTINUOUS SLABS

Both field observations and laboratory test programs have shown a definite enhanced punching capacity in continuous concrete slab systems, beyond the established code strength values.

At the MILL RACE high explosive test event (Ref. 8), a representative shored, continuous slab structure was subjected to the loading of a 40 psi overpressure. The loading was of sufficient intensity to develop flexural cracking; however, there was

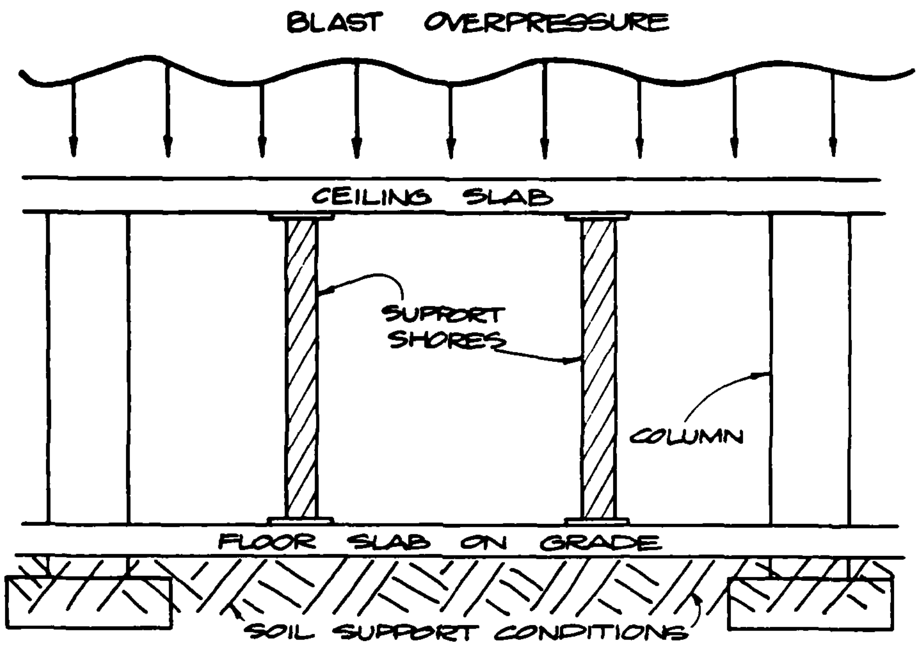


Fig. 5-1a. Shored Slab System.

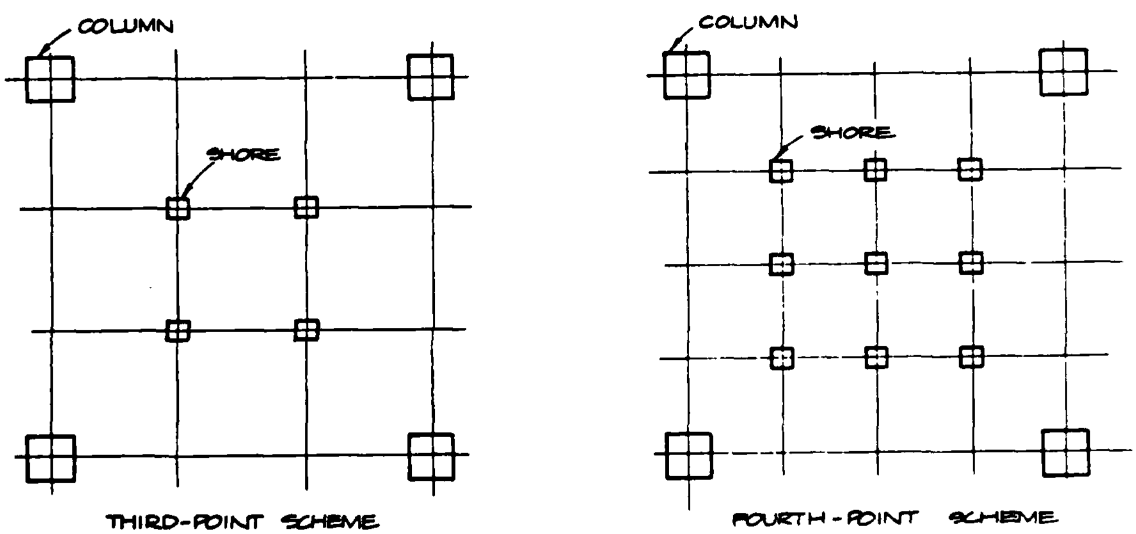


Fig. 5-1b. Shoring Configuration.

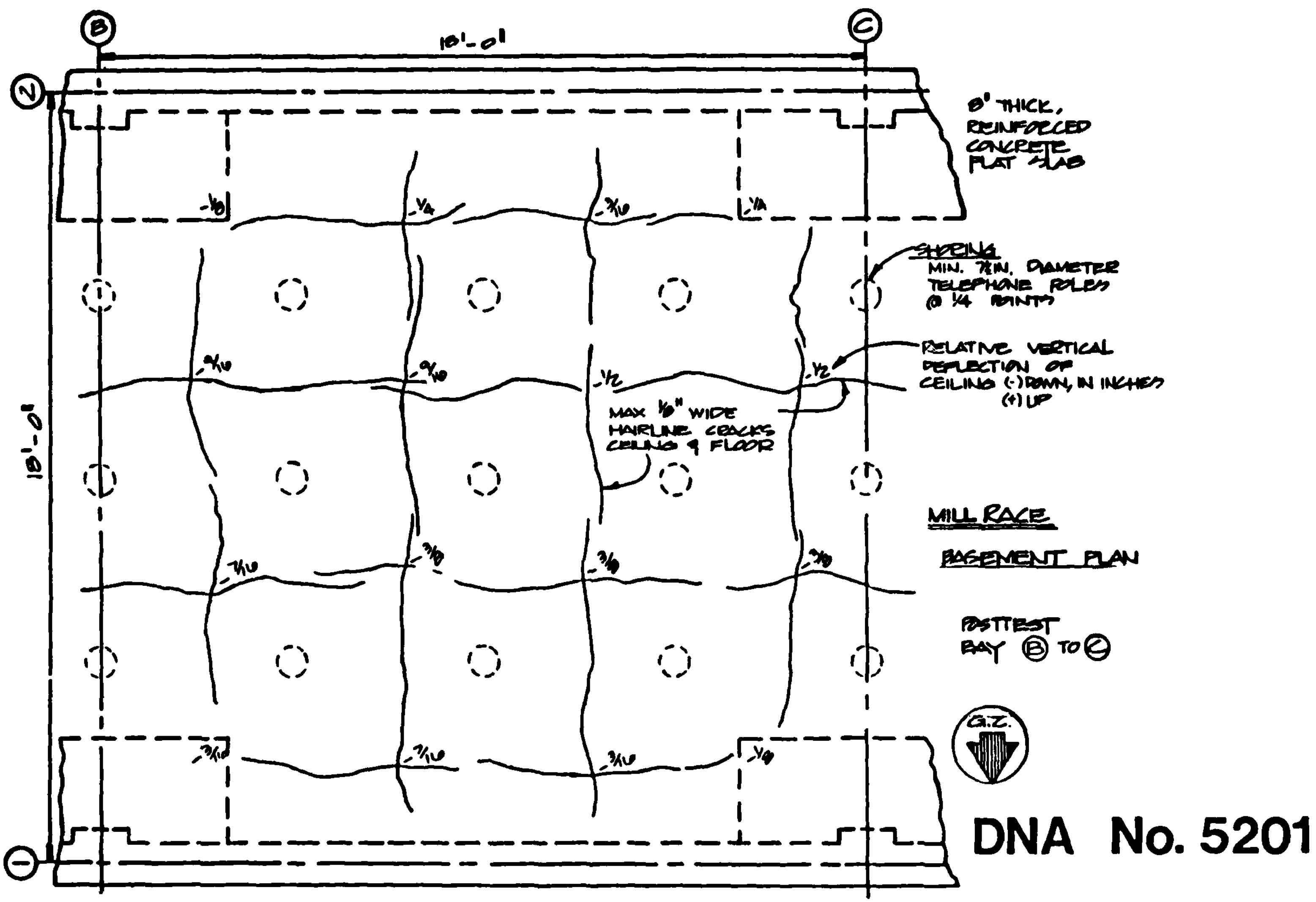


Fig. 5-2. MILL RACE Slab Cracking Pattern.

no indication of punching shear cracking at any of the shore locations, as shown in Figure 5-2 (from Ref. 8).

Previously conducted laboratory tests include a series of isolated (non-continuous) slab elements (Ref. 1) containing either top or bottom reinforcing steel, which were loaded to failure with various types of shores. Regardless of the location of the steel in the test slabs, the failure loads were approximately 50% to 70% greater than predicted by existing building code procedures. This strength increase was primarily the result of the load distribution effect of the reinforcing steel layers, since arch action was not effective because of the size of the slab elements. In the actual continuous building slab, additional strength is provided by both arch action and the distribution of the shore loading (bearing plate distribution action) by the reinforcing steel layers.

In view of these field and laboratory test results, a concentrated literature search was performed in order to develop a strong theoretical and experimental justification for the compression membrane, or arch action, effect on punching shear capacity.

LITERATURE REVIEW

In a plain or reinforced concrete plate, radically different strength behavior occurs depending upon whether the plate edges are laterally constrained or simply supported. Both the flexural and punching capacity are significantly enhanced by the arching or compressive membrane action in slabs with fixed boundaries. Ref. 20 provides a good historical review, beginning with Westergaard and Slater who observed this effect in tests of flat slab floor panels conducted early in this century. Later, other researchers, Ockleston, Christiansen, Wood and Park (Ref. 20) developed more tests and theories; and also other related work is given in Refs. 21 through 29, some of which will be discussed later. The overall results and conclusions in the literature confirm the existence of arch-action strength, and present simple means of estimating the additional strength.

For restrained slab punching shear strength, Ref. 23 states:

"ACI Code 318-77 [Ref. 6] assumes that the punching shear strength of concrete is $4\sqrt{f'_c}$ for a critical section located half the effective depth of the slab, d/2, outside the perimeter of the loaded area. ACI-ASCE Committee 426 has reported that, in addition to the concrete compressive strength, f'_c, six variables also affect the punching shear strength: (1) the ratio between the side length of the loaded area and the effective depth of the slab, c/d, (2) column shape, (3) rate of loading, (4) type of concrete, (5) ratio of the shear to flexural capacity for the slab, ϕ_o, and (6) boundary restraints or prestress for the slab."

With respect to item (6), "in-plane compression", Ref. 23 continues,

"The effects of the ratio of the shear to flexural strength, ϕ_o, in-plane restraints, prestress rotational demands and reinforcement ratio are interrelated problems. Inclined cracking develops at about the same shear stress for either a wide beam or punching shear failure. However, for the punching situation those cracks cannot open until there is a marked decrease in the tangential stiffness of the slab. A two-way reinforcement pattern or in-plane restraints will maintain stiffness and permit development of an ultimate capacity considerably greater than the wide beam capacity. If a slab is properly designed according to ACI 318-77 [Ref. 6] concepts the flexural strength should be slightly less than the shear strength and thus the 318-77 provisions attempt to define the punching shear strength for the onset of large rotations. That condition is conservatively presumed to correspond to a ϕ_o value of unity since for a ϕ_o value at failure less than unity the shear strength exceeds $4\sqrt{f'_c}$ . . . .

"Horizontal in-plane restraints provided by an external frame or the presence of an elastic area in the slab surrounding a yielding area considerably enhance the capacity of a slab for which the ϕ_o value is close to unity in the unrestrained state." [See Figures 5-3a and 5-3b

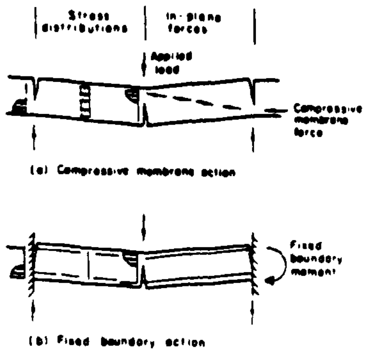


Fig. 5-3a. One-Way Slab Wedges

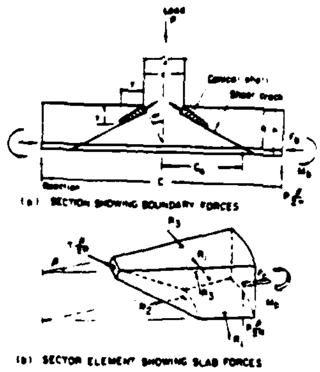


Fig. 5-3b. Two-Way Slab Wedges

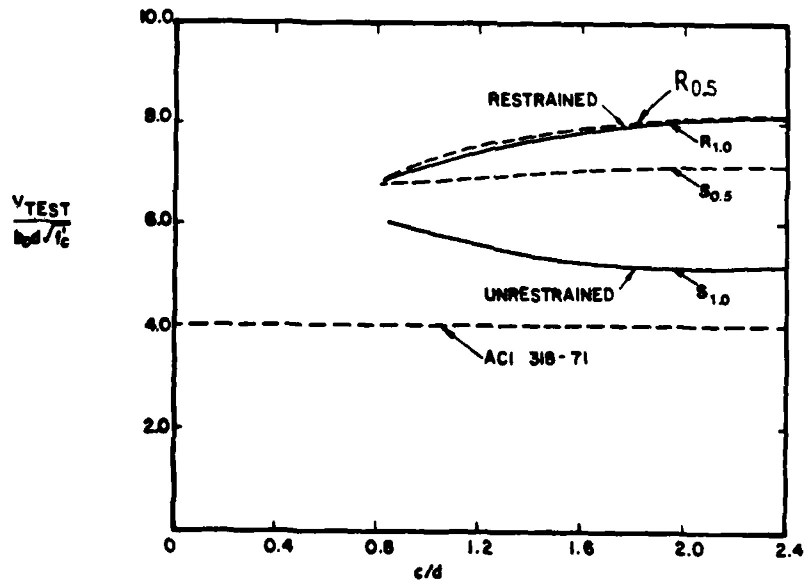


Fig. 5-3c. Effect of In-Plane Constraint on Shear.

from Ref. 26] "While such restraints increase the shear strength they also substantially reduce the ductility at failure. The effects of in-plane restraints on shear strength are illustrated in Fig. 4 Figure 5-3c . . . . The curve labelled S_1 indicates the variation in shear strength with c/d for a simply supported slab having a ϕ_o value of approximately unity. The curve $S_{0.5}$ indicates the same result for a ϕ_o value of about 0.5. The curves labelled R_1 and $R_{0.5}$ indicate results for the same specimens with in-plane deformations restrained by an extremely strong surrounding steel frame. A model for determining the degree of restraint and its effect on shear strength has been developed by Hewitt and Batchelor [Ref. 26]."

Figures 5-3a and 5-3b from Ref. 26 show the mechanical model of the constrained slab.

"Prestressing the slab provides an initial in-plane compression. The shear stress for failure then becomes that causing a principal tensile stress of about $4\sqrt{f'_c}$ at mid-depth of the slab. Further, as for external restraints, the ductility decreases as the prestress level increases."

A simple model of these effects and strength equations are given in Ref. 24, by Rozvany and Hampson.

ADAPTATION OF ARCHING ACTION THEORY TO PUNCHING STRENGTH

Ref. 29 provided an extensive study and analysis of the role of arching action in normal-to-plane blast loading of constrained wall panels. The formation of cracked, wedged segments in a shored slab at the shore-slab intersection is analogous to the constrained wall panel conditions; compare Figures 5-4 and 5-5 for walls with Figure 5-3a for slabs. Primarily, the slightly deformed cracked segment configuration, as it becomes wedged between the constraining frame beams, creates an in-plane compression condition because of the arch-thrust, see Figure 5-6. This compression can significantly increase the punching shear capacity by changing the

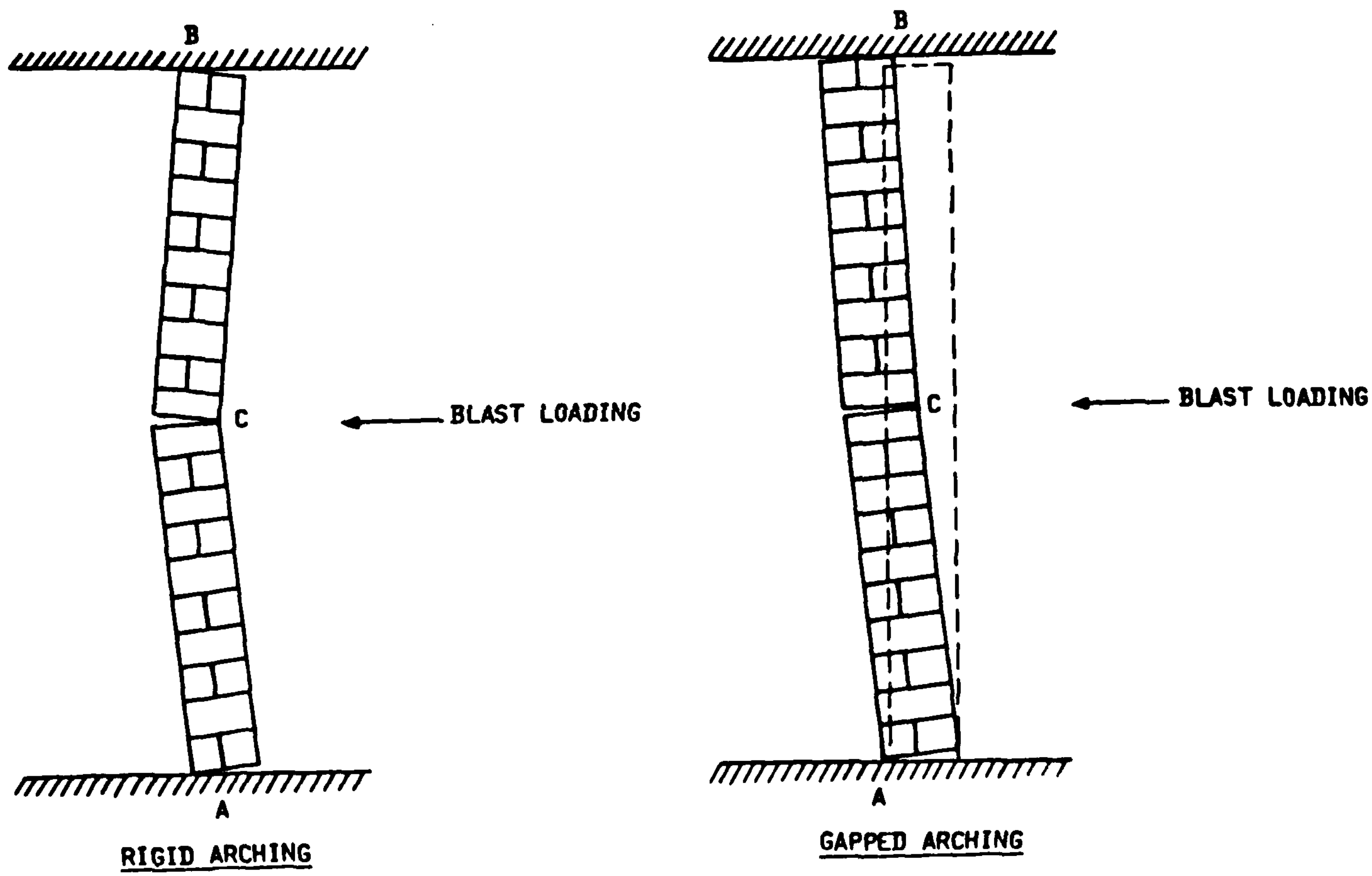


Fig. 5-4. Arching Mechanisms in Walls.

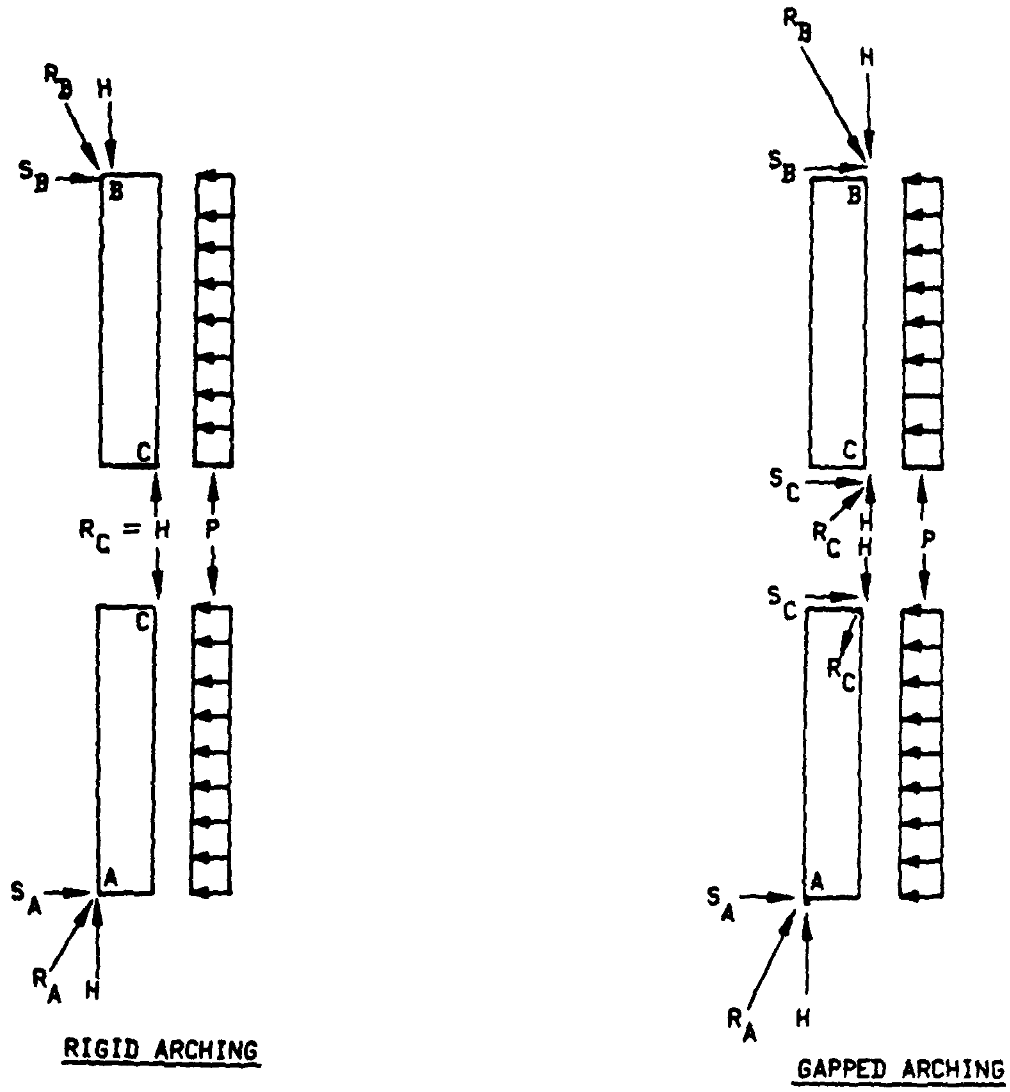


Fig. 5-5. In-Plane Compression Conditions.

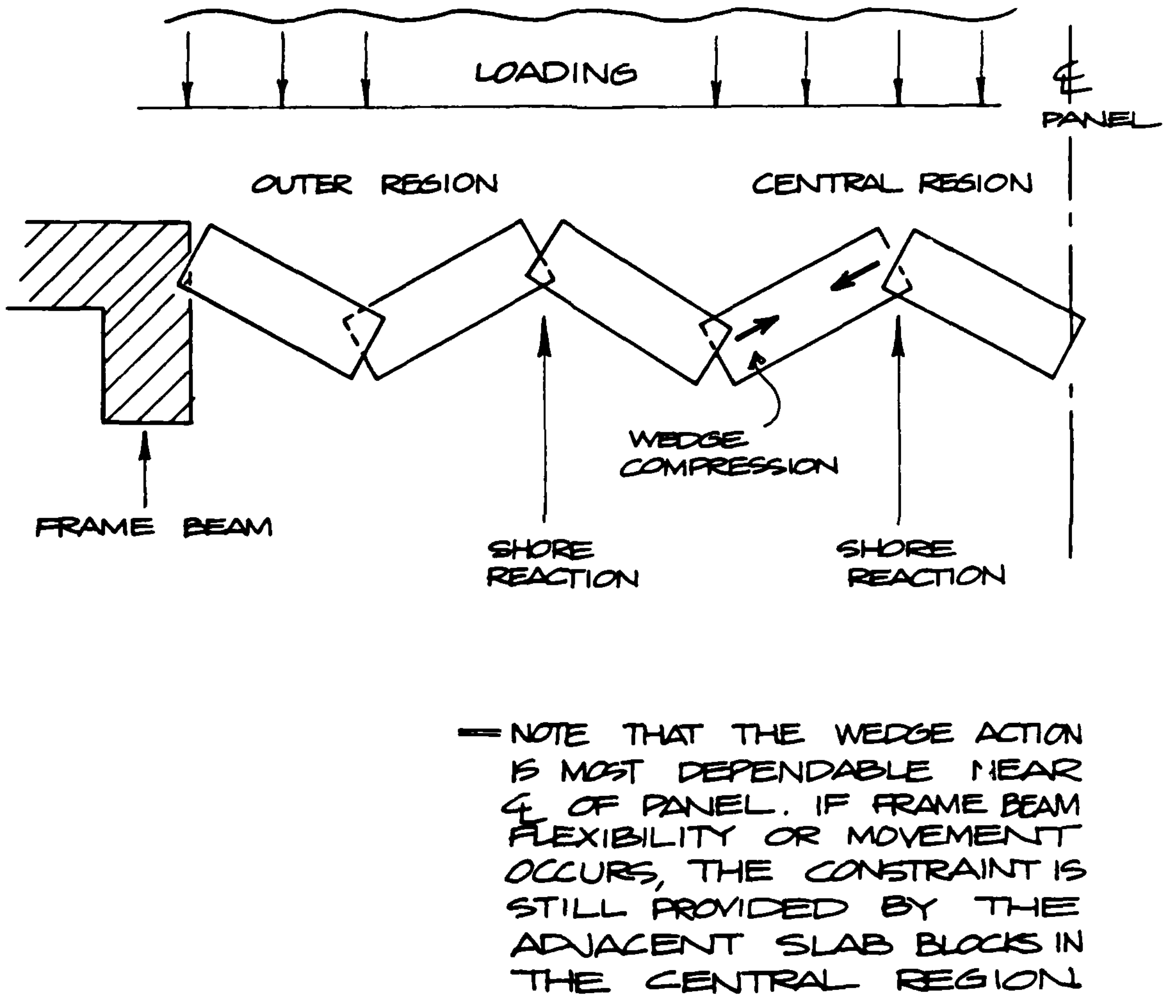


Fig. 5-6. Arching or Wedge Mechanism in Shored Slabs.

ıf principal tension cracking. This plane is at approximately 45 degrees compression, and will make a lesser angle (about 20 degrees) with the tal plane with compression. See Figure 5-7.

simple model of the net physical effect is that the perimeter of the area for ıg shear stress at $4\sqrt{f'_c}$ is increased from a distance of t/2 (where t = slab ːss) from the shore bearing area to a distance of t, or possibly even greater, he shore bearing area, depending on the intensity of the confining compressive This intensity is high in the central portion of the slab (Section B-B of 5-8) when typical shore spacing of 4 ft to 6 feet is used.

UATION OF PUNCHING SHEAR CAPACITY

With reference to Figure 5-8, a typical layout of shores in a slab panel is . Section A-A shows a shore in the negative moment region where top ɪrcing steel is present. Here, it could be posssible that compressive arch ı may be weakened by the frame or beam movement, or by non-symmetrical ıg (also see Figure 5-6). However, the top steel assists in providing good ing shear resistance (see Ref. 23) even where the arch-action is small.

Section B-B shows the central location where top steel may not be present; and ut the compressive effects of arch action, the slab punching capacity would be However, since this central portion of the slab area is strongly confined, or ɾained, by the outer negative moment regions, there exists a dependable amount ːh action, and the punching strength is enhanced. The presence of the bottom ːive moment) steel also helps to distribute the shore reaction.

For the purpose of evaluating the punching shear capacity, it is recommended a punching shear stress of $4\sqrt{f'_c}$ be used with a perimeter that varies as the of the distance between shores (ℓ) to the slab thickness (t). This method of ation takes into account the enhancement of the punching strength through arch n when the shores are closely spaced, and decreases the significance of this ıcement as the shore spacing increases.

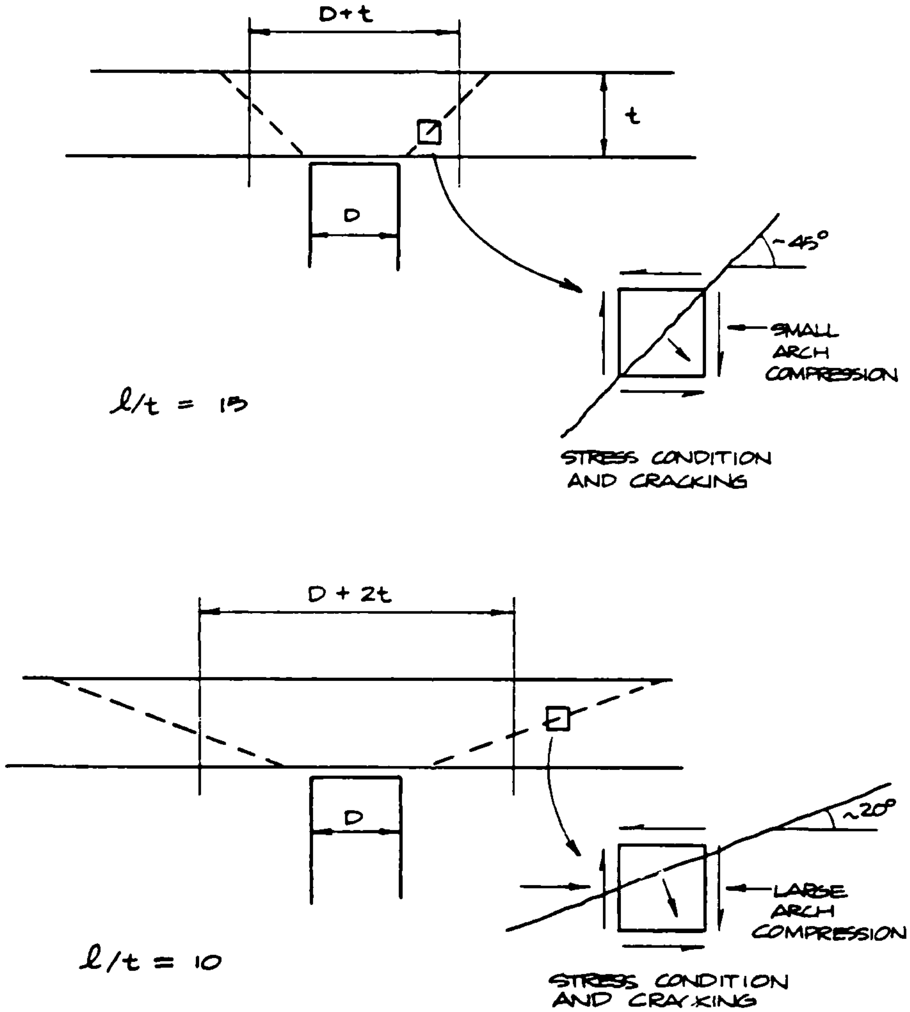


Fig. 5-7. In-Plane Compression Effects.

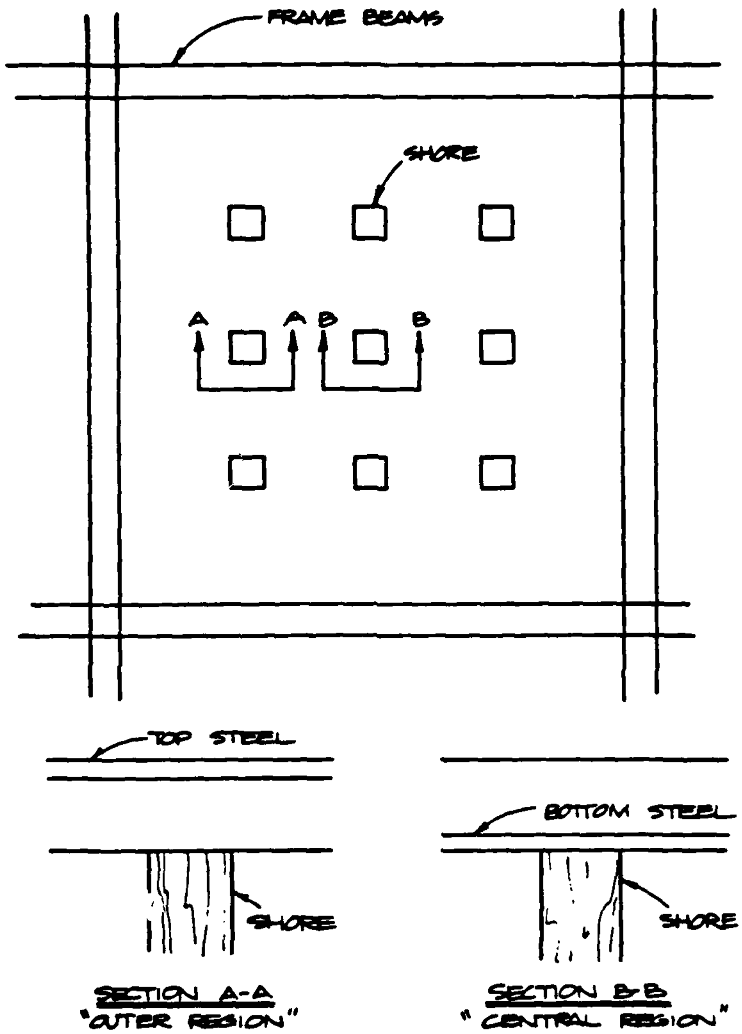


Fig. 5-8. Slab Reinforcing Steel Conditions.

Determination of the perimeter by this method is accomplished by modification of the standard ACI formulas to include a k factor where

$$k = 4 - 1/5\ \ell/t$$

Thus, the relationship may be presented as,

$$V_p = 4\sqrt{f'_c}\ 4(D + kt)t$$

where V_p is normal shear strength provided by the concrete, lb
f'_c is compressive strength of concrete, psi
D is width or diameter of bearing, in. (shore or cap plate)

This relationship appears to be valid for an ℓ/t ratio between the values of 8 and 20, which includes the range of building spans that might be expected to require upgrading. A pictorial representation of several of the parameters used in this relationship is shown in Figure 5-9.

In order to further expand this evaluation for use in shelter upgrading, V_p may be divided by the effective area (tributary area less the punching perimeter), thus providing the overpressure, p_o, at which shore punching becomes a critical failure mode.

$$p_o = \frac{4\sqrt{f'_c}\ 4(D + kt)t}{\ell^2 - (D + kt)^2}$$

APPLICABILITY TO FLOOR SLABS ON GRADE

While the arching theories and related tests have been primarily concerned with slabs on column supports, they can also be applied directly to the punching resistance for floor slabs on grade. The typical slab on grade is four or more inches in thickness, reinforced with welded-wire fabric or minimum reinforcing steel for crack control, and is placed on a reasonably firm base material.

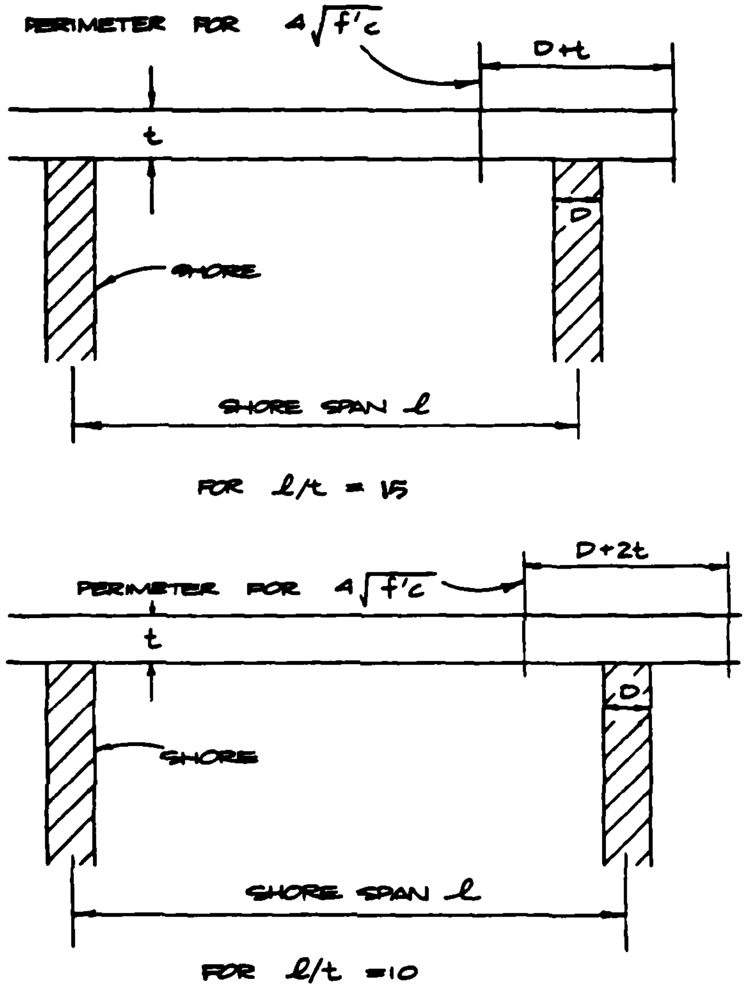


Fig. 5-9. Parameters for Evaluation of Punching Strength.

With distances between shores of 4 to 6 feet, it is reasonable to assume that arching action can develop the compressive strengths necessary for extra punching shear strength, see Figure 5-10. It is felt that this extra resistance is also enhanced by the presence of the grid reinforcing, and the supporting effect of the sub-grade material under the shore bearing area. Further, these floor slabs are usually well confined by wall footings, and are not subject to non-symmetrical or lateral load effects. Therefore, arching action is dependable throughout the floor slab area. With these extra effects, the punching capacity of a floor slab is equal or superior to the floor or roof slab for typical construction configurations and shore spacing.

CONCLUSIONS

An approximate method of evaluating the punching shear capacity has been provided, and all indications are that this method is the relationship necessary to predict critical shore punching for non-typical, as well as typical, shore spacing.

As a result of this investigation, as well as the data developed from past analyses and testing, it can be concluded that in most cases punching shear would not be the critical mode of failure. It is more probable that failure will occur as a result of connection fracture, shore failure, or slab flexure — all areas that are currently, or will be, the subject of further research.

At the present time, we do not anticipate the need for conducting additional test programs on shore punching of reinforced concrete continuous slabs or slabs on grade. Instead, the emphasis this year will be directed toward the further development of the relationship presented herein, and the presentation of this relationship in a usable format (graphs or charts) for inclusion in the shelter upgrading manuals now under development.

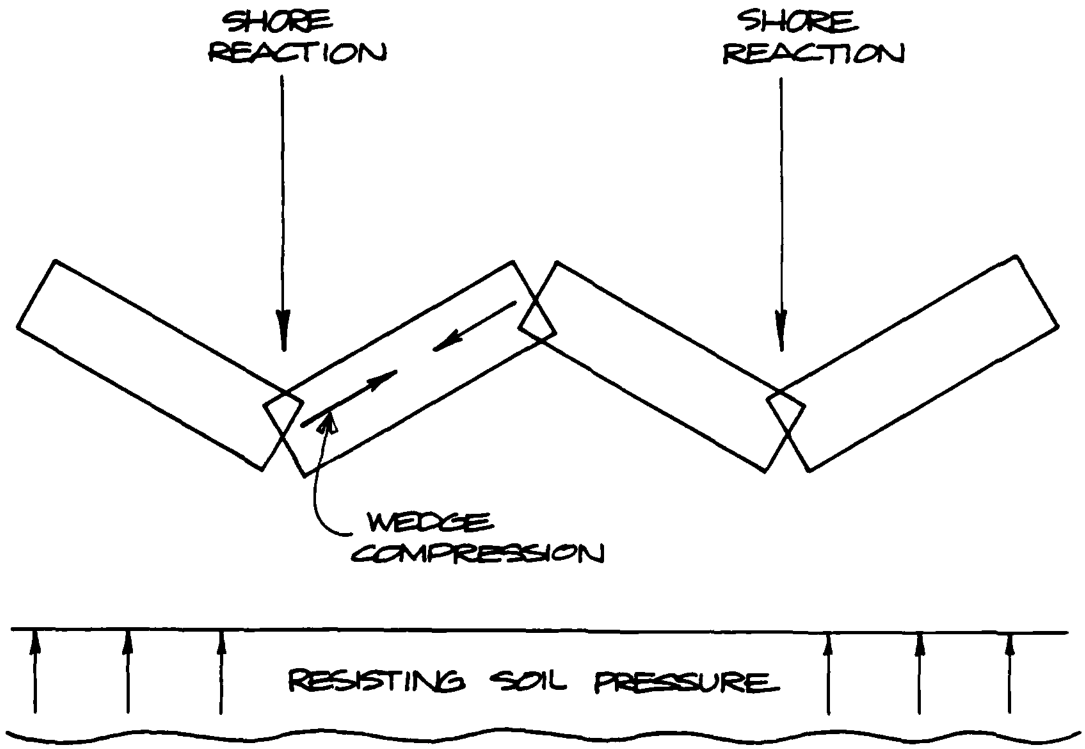


Fig. 5-10. Floor Slab Arching Mechanism.

Section 6

PROGRAM SUMMARY AND RECOMMENDATIONS

This section presents a summary to date of the work conducted during the initial phase of a five-year program directed toward the development of the information and methodologies required for the selection and upgrading of existing structures for use as shelter spaces. Upon completion, the results of the program will be presented in the form of a survival matrix that will permit rapid analysis and overall assessment of protective capabilities, and manuals presenting the plans and methods in a user-guide form suitable for use by planners in the field.

The five-year program was divided into five basic tasks:

Task 1 - Manual Development

Task 2 - Prediction Methodology

Task 3 - Training and Survey Procedures

Task 4 - Field Testing

Task 5 - Analysis and Testing

The first year's research effort proposed to include portions of each of these basic tasks, and, in general, the investigation was accomplished in this manner. Based on periodic evaluations, and in order to provide efficiency and continuity, particularly within the analysis and testing task, several of the proposed subtasks required modification and redirection.

Both the existing key worker and host area shelter manuals were reviewed for the purpose of identifying areas that required additional research, simplification, and/or modification. These reviews generated specific comments concerning manual organization, shelter closures and entranceways, resource requirements, ventilation criteria, expedient shelter designs, and fallout protection.

The development of a prediction methodology for both manuals concentrated primarily on the analysis of building types, laboratory and field tests, and analytical work on structural components for inclusion in the various indices.

In the areas of training and survey, a review was conducted of the available FEMA guidance, photographs, and data on upgrading techniques and expedient shelters. The Shelter Survey Technician Course, developed by FEMA, was reviewed and critiqued.

Full-scale static load tests to failure were conducted on two sizes of glulam beams. Four tests on beams 6-3/4 in. wide and 16-1/2 in. deep, two unshored (base cases) and two shored at midspan, were conducted resulting in failure loads of 1.1 psi for the unshored to 4 psi for the shored. Three tests on 3-1/8 in. wide by 18 in. deep beams were conducted, one unshored and two shored at midspan, with failure loads ranging from 0.6 psi for the unshored to 1.7 psi for the shored. These tests were conducted with two sizes of bearing plates in order to obtain additional data. An extensive analysis using these test data, supplemented by data from other sources, was conducted for the purpose of developing prediction parameters for these types of timber members. This analysis is presented in an appendix of this report.

Three static/dynamic drop tests were conducted on prestressed precast concrete plank. These planks were similar to those tested statically in a previous program. The test results verified that dynamic load test results could be predicted using the data obtained from static load tests on identical specimens.

The effort this year with respect to punching shear was directed toward the investigation of the increase in punching resistance in shored concrete slabs over the recognized code value. This increased value has been observed in both laboratory and field tests. Research has indicated that this enhanced resistance is theoretically valid, and is primarily the result of the arch-action effect found in constrained slabs.

The first phase of this program accomplished the majority of the intended basic goals, and provided the data base required for continuation of the program into the

second year's effort with continuity. The manual development task will continue in the revision and updating of both manuals, and the incorporation of additional data. The prediction methodology will be under continuous development, with additions and modifications to the various indices, and the formulation of workable index formats. A prototype training package will be developed based on the findings of the first year's review of available material, and any formats developed for the prediction index will be field tested to determine their viability.

With respect to the analysis and testing task, the first year's effort required some modification and redirection. Some of the analytical work resulted in a more extensive investigative effort than anticipated, and several of the testing programs were enlarged and extended to additional related areas, resulting in more efficiency and economy in the overall program. Other programs were delayed in order to make better use of our test facilities, and one investigation resulted in our recommending that no further test effort be expended during the second year.

The comprehensive analysis and testing program on glulam beams was extended to the evaluation of glulam connections by expedient use of sections of previously tested and failed beams, and will be completed in the second year. The tests on the roof truss systems is substantially complete, but will not be reported at this time as the program has been extended in an attempt to obtain data on the long-duration load effects. In conjunction with this test program, tests are also underway on timber panelized roof grid systems, using both short- and long-duration loading.

A complete survey and analysis was completed and reported on concrete structural connections, and the second year's effort will be directed toward steel and timber connections.

A test program on closure types is currently underway in conjunction with Ballistic Research Laboratory, and it is anticipated that the results will be available during the second year.

The investigations directed at the analysis and testing of various wall types, with and without fallout protection, were delayed in anticipation of obtaining the use

of test facilities that would lend themselves to a more efficient test schedule and use of materials and manpower. These investigations will be conducted during the second year.

The analysis of punching shear in concrete slabs resulted in a recommendation that the second year's effort be postponed. During this year's effort an approximate method of evaluating the punching shear capacity has been developed, and all indications are that this method is the relationship necessary to predict critical shore punching for non-typical, as well as typical, shore spacing. As a result of this investigation, as well as the data developed from past analysis and testing, it can be concluded that in most cases punching shear would not be a critical mode of failure. It is more probable that failure will occur as a result of connection fracture, shore failure, or slab flexure — all areas that are currently the subject of research under this program.

Tables 6-1 and 6-2 are the preliminary survival matrices for floor and roof systems, respectively. These matrices were initially published in SSI Report No. 7910-5 (Ref. 2), and have been continually updated to include current data. The results of the testing and analysis conducted during the initial phase of this program, as reported herein, are included, as well as the data developed at the MILL RACE event (Ref. 8) and from tests conducted at the Waterways Experiment Station. These matrices indicate the overpressure in psi at which 95% of the floor or roof systems are predicted to survive "as built" and with various types of shoring. Those that have been tested are also indicated in the tables.

The survival pressures indicated for the various types of construction were determined by assuming the dead loads (load of structure itself) and increasing the design live loads by the safety factors required for design, as outlined in the applicable codes, for the particular construction considered. The "as built" survival overpressure considers the floor or roof "as is" with no upgrading or shoring, and ALL assume radiation protection equal to a P_f of 100 (18 inches of earth equivalent) superimposed on the floor.

TABLE 6-1: PRELIMINARY SURVIVAL MATRIX FOR FLOORS

Overpressures at which 95% of Floors Will Survive "As Built" and with Various Types of Shoring (All Values in psi)

Type of Floor Construction and Dead Load	As Built		Shoring Required: Midspan		1/3 Span		1/4 Span		King-Post Truss		Flange		Boxed Beam	
	Pred	Test	Pred	Test	Pred	Test	Pred	Test	Pred	Test	Pred	Test	Pred	Test
WOOD														
Light - Joist	†	0.4	3.3	6.8	8.6	9.2	-		1.6	1.8	1.1	1.1	1.1	2.1
Light - Glulam	†	0.2	3.3	3.0	8.6									
Medium - Joist	0.9	1.5	6.7	8.1	16.4	12.9	-		3.8		2.8		2.8	
Medium - Glulam	0.9	1.3	6.7	7.0	16.4		-							
Heavy - Plank	2.6		13.7		32.1				8.2		-		-	
STEEL, LIGHT														
Light - Open-Web Joist	0.2	0.3	1.0	1.2	2.8	3.5	-		1.0		-		-	
Medium - Open-Web Joist	1.4	1.6	3.0	3.4	6.6	8.0	-		2.5		-		-	
STEEL, HEAVY														
Light - Beam and Slab	0.1		3.1		7.9		-		-		-		-	
Medium - Beam and Slab	0.8		5.5		13.3		-		-		-		-	
Heavy - Beam and Slab	2.0		10.3		24.0		-							

Note: Overpressure values assume radiation protection equal to a P_f of 100 (18 in. of earth or equivalent) superimposed on floor. Assumed density of earth = 100 pcf.

†- Required radiation protection (P_f = 100) would cause floor to collapse.

TABLE 6-1: PRELIMINARY SURVIVAL MATRIX FOR FLOORS (contd)

Overpressure at which 95% of Floors Will Survive "As Built" and with Various Types of Shoring (All Values in psi)

			Shoring Required					
Type of Floor Construction and Dead Load	As Built		Midspan		1/3 Span		1/4 Span	
	Pred	Test	Pred	Test	Pred	Test	Pred	Test
CONCRETE								
Light - Single and Double Tees, One-Way Joists	0.6		4.7		11.7		-	
Light - Hollow-Core Slabs	0.6	*0.6*	4.8	*6.2*	11.7	*16.1*	-	
Light - One-Way Solid Slabs	0.9		5.0		12.0		-	
Light - Flat Slab, Flat Plate - Two-Way	0.9		5.0		12.0		21.7	
Light - Waffle Slab	0.6		4.7		11.7		21.4	
Medium - Single and Double Tees, One-Way Joists	1.3		7.6		18.0		-	
Medium - Hollow-Core Slabs	1.5	*2.2*	7.8	*10.2*	18.2	*19.8*	-	
Medium - One-Way Solid Slabs	1.7		8.0		18.4		-	
Medium - Flat Slab, Flat Plate - Two-Way	1.7		8.0		18.4		33.0	*36.5* (1) *37.0* (2)
Medium - Waffle Slab	1.3		7.6		18.0		32.6	*43.0* (1)

Note: Overpressure values assume radiation protection equal to a P_f of 100 (18 in. of earth or equivalent) superimposed on floor. Assumed density of earth = 100 pcf.

(1) Waterways Experiment Station Test

(2) MILL RACE Test

TABLE 6-1: PRELIMINARY SURVIVAL MATRIX FOR FLOORS (contd)

Overpressure at which 95% of Floors Will Survive "As Built" and with Various Types of Shoring (All Values in psi)

			Shoring Required					
Type of Floor Construction and Dead Load	As Built		Midspan		1/3 Span		1/4 Span	
	Pred	Test	Pred	Test	Pred	Test	Pred	Test
CONCRETE								
Heavy - Single and Double Tees, One-Way Joists	2.8		13.2		30.6		-	
Heavy - Hollow-Core Slabs	3.0	*5.3*	13.4	*19.9*	30.8	*30.1*	-	
Heavy - One-Way Solid Slabs	3.3	*5.9*	13.7	*17.7*	31.1	*36.2*	-	
Heavy - Flat Slab, Flat Plate - Two-Way	3.3		13.7		31.1		55.4	
Heavy - Waffle Slab	2.8		13.2		30.6		54.9	

Note: Overpressure values assume radiation protection equal to a P_f of 100 (18 in. of earth or equivalent) superimposed on floor. Assumed density of earth = 100 pcf.

TABLE 6-2: PRELIMINARY SURVIVAL MATRIX FOR ROOFS

Overpressure at which 95% of Roofs Will Survive
'As Built" and with Various Types of Shoring. (All Values in psi)

		Shoring Required							
pe of Roof nstruction	Load① Rating	As Built		Midspan		1/3 Span		1/4 Span	
		Pred	Test	Pred	Test	Pred	Test	Pred	Test
WOOD									
t	A	†	-	0.7	-	-	-	-	-
	B	†	-	1.4	2.0 ②	-	-	-	-
	C	†	-	2.8	-	-	-	-	-
am	A	†	†	0.7	0.5	-	-	-	-
	B	†	†	1.4	1.2	-	-	-	-
	C	†	0.3	2.8	2.8	-	-	-	-
ed Truss	A	†	†	†	†	0.2	0.2 ③	0.7	1.1
	B	†	†	0.2	0.5	0.8	1.1	1.4	-
	C	†	0.3	0.8	1.8	1.8	2.9	2.4	-
IGHT STEEL									
-Web Joist Plywood Deck	A	†	-	†	-	0.2	-	-	-
	B	†	-	†	-	0.7	-	-	-
	C	†	-	0.3	-	1.6	-	-	-
EAVY STEEL									
-Web Joist Metal Deck	A	†	-	†	-	0.9	-	-	-
	B	†	-	0.2	-	1.4	-	-	-
	C	†	-	0.6	-	2.3	-	-	-

Overpressure values assume radiation protection equal to a P_f of 100 (18 in. of earth or equivalent) superimposed on roof. Assumed density of earth = 100 pcf.

† - Required protection (P_f = 100) would cause roof to collapse.

① - See Table 6-3.

② - MILL RACE Test.

③ - Waterways Experiment Station Test.

If roof construction is concrete, use Floor Matrix, Light Concrete Construction.

Table 6-2, the survival matrix for roofs, has been revised considerably in format in order to increase its accuracy and usefulness. The lower portion of the matrix, which covered roofs constructed of various concrete systems, has been deleted, and a note added to direct the user to use the floor matrix, Table 6-1, for "light" concrete construction to determine the survivability of concrete roofs. The reason for this modification is that, because of construction and material costs, concrete roof systems are rarely designed for only light roof loadings. When these systems are used as roofs, the roof is many times designed as a future floor, for mechanical loads, or located in an area with severe snow load requirements. Accordingly, it was determined that the use of the "light" concrete floor matrix for concrete roofs was appropriate and conservative, and provided a simplification for the user.

The other revision to the roof survival matrix consisted of adding an additional column headed "Load Rating". The purpose of this revision was to incorporate into the matrix the increased values of survivability that might be expected for roofs in locations where snow loads are required by code to be incorporated into the design. Snow loads are required, with some modification, to be added to the design live loads and in many areas of the country would significantly enhance the survival of roof systems. This revision was not technically possible until the completion of this year's effort, since our previous programs did not include any roof tests for verification of the prediction methodology. The tests conducted this year on glulam roofs and the gabled roof truss systems, as well as the results from MILL RACE, provided the required data base.

The designated ratings, "A", "B", and "C" refer to three values of loads that include a constant live load, 16 psf, plus minimum snow loads of 0 psf, 10 psf, and 30 psf, respectively, each reduced by a basic snow load coefficient as per design requirements. These ratings permit the matrix user to more accurately determine a roof system's performance by including a geographical parameter. In actuality, the snow design loads in the United States vary from zero to greater than 80 psf, with some exceeding 200 psf in high mountainous areas. However, the great majority of the population centers can be conservatively grouped into these three snow load categories, as shown on Table 6-3.

TABLE 6-3

LOAD RATINGS FOR SELECTED AREAS FOR USE WITH ROOF MATRIX

A	B	C
Los Angeles, CA	New York City	Minnesota
San Francisco, CA	Illinois	Buffalo, NY
Texas	Pennsylvania	Rochester, NY
Phoenix, AZ	Detroit, MI	Albany, NY
Georgia	Massachusetts	Vermont
Florida	Indiana	New Hampshire
Alabama	Ohio	Maine
Mississippi	Kentucky	Colorado
Arkansas	West Virginia	Alaska
San Diego, CA	Virginia	Montana
San Jose, CA	Washington, DC	Utah
Louisiana	Missouri	Wisconsin
Sacramento, CA	Maryland	Wyoming
Las Vegas, NV	New Jersey	Idaho
Memphis, TN	Milwaukee, WI	
Hawaii	Kansas	
South Carolina	Nebraska	
Tucson, AZ	Tennessee	
	Rhode Island	
	Connecticut	
	Delaware	
	Seattle, WA	
	Iowa	
	Reno, NV	
	South Dakota	
	Albuquerque, NM	
	North Carolina	
	North Dakota	
	Oklahoma	
	Portland, OR	

REFERENCES

1. Tansley, R.S., B.L. Gabrielsen, and G.J. Cuzner, **Upgrading of Existing Structures Phase III: Shelter Design Options,** SSI 8012-6, Scientific Service, Inc., Redwood City, CA, May 1981.

2. Gabrielsen, B.L., R.S. Tansley, and G. Cuzner, **Upgrading of Existing Structures: Phase II,** Scientific Service, Inc., Redwood City, CA, June 1980.

3. **Manufacturing Standard Specifications for Structural Glued Laminated Timber of Softwood Species (AITC 117-79),** American Institute of Timber Construction, Englewood, CO, 1979.

4. **Voluntary Standard of Timber Construction for Structural Glued Laminated Timber (PS 56-73),** American Institute of Timber Construction, Englewood, CO, 1976.

5. **Uniform Building Code,** International Conference of Building Officials, Whittier, CA., 1979 edition.

6. **Building Code Requirements for Reinforced Concrete (ACI 318-77),** American Concrete Institute, Detroit, MI, 1977.

7. Large, G.E., and T.Y. Chen, **Reinforced Concrete Design,** The Ronald Press Company, New York, NY, 1969.

8. Tansley, R.S., and J.V. Zaccor, **Testing of Shelter Design and Industrial Hardening Concepts at the Mill Race Event,** SSI 8115-4, Scientific Service, Inc., Redwood City, CA, January 1982.

9. Gabrielsen, B.L., G. Cuzner, and R. Lindskog, **Blast Upgrading of Existing Structures,** SSI 7719-4, Scientific Service, Inc., Redwood City, CA, January 1979.

10. McVay, M.K., **Test and Analysis of Upgraded One-Way Reinforced Concrete Floor Slabs,** U.S. Army Waterways Experiment Station, Vicksburg, MS, July 1981.

11. **CRSI Handbook,** Concrete Reinforcing Steel Institute, Chicago, IL, 1980.

12. **Data Package for a Nine-Bay, 1/4-Scale Model of an Upgraded Flat Plate and An Upgraded Two-Way Slab with Beams,** U.S. Army Waterways Experiment Station, Vicksburg, MS, March 1982.

13. **Post-Tensioning Manual,** Post-Tensioning Institute, Phoenix, AZ, 1972.

14. Leyendecker, E.V., and G.S. Fattal, **Investigation of the Skyline Plaza Collapse in Fairfax County, Virginia,** National Bureau of Standards, Center for Building Technology, Gaithersburg, MD, 1973.

15. Phillips, W.R., and D.A. Sheppard, **Plant Cast, Precast and Prestressed Concrete, A Design Guide,** The Prestressed Concrete Manufacturers Association of California, 1980.

16. **PCI Design Handbook - Precast, Prestressed Concrete,** Second Edition, Prestressed Concrete Institute, Chicago, IL, 1978.

17. **PCI Manual on Design of Connections for Precast Prestressed Concrete,** Prestressed Concrete Institute, Chicago, IL, 1973.

18. Mast, R.F., "Roof Collapse at Antioch High School," **Journal of the Prestressed Concrete Institute,** Vol. 26, No. 3, May-June 1981.

19. **PCI Manual for Structural Design of Architectural Precast Concrete,** MNL-121-77, Prestressed Concrete Institute, Chicago, IL, 1977.

20. Brotchie, J.F., and M.J. Holley, "Membrane Action in Slabs," **Cracking, Deflection, and Ultimate Load of Concrete Slab Systems,** ACI Publication SP-30, American Concrete Institute, Detroit, MI, 1971.

21. Hopkins, D.C., and R. Park, "Test on a Reinforced Concrete Slab and Beam Floor Designed with Allowance for Membrane Action," **Cracking, Deflection, and Ultimate Load of Concrete Slab Systems,** ACI Publication SP-30, American Concrete Institute, Detroit, MI, 1971.

22. Tong, P.Y., and B.deV. Batchelor, "Compressive Membrane Enhancement in Two-Way Bridge Slabs," **Cracking, Deflection, and Ultimate Load of Concrete Slab Systems,** ACI Publication SP-30, American Concrete Institute, Detroit, MI, 1971.

23. Hawkins, N.M., and D. Mitchell, "Progressive Collapse of Flat Plate Structures," **ACI Journal** No. 7, Proc. V.76, July 1979.

24. Rozvany, G.I.N., and A.J.K. Hampson, "Optimum Design of Prestressed Plates," **ACI Journal,** No. 8, Proc. V.60, August 1973.

25. Gazetas, George, and T.P. Tassios, "Elastic-Plastic Slabs on Elastic Foundation," **Journal of the Structural Division,** ASCE, Vol. 104, No. ST4, Proc. Paper 13704, April 1978.

26. Hewitt, B.B., and B.deV. Batchelor, "Punching Shear Strength of Restrained Slabs," **Journal of the Structural Division,** ASCE, Vol. 101, No. ST9, Proc. Paper 11548, September 1975.

27. Braestrup, M.W., "Dome Effect in RC Slabs: Rigid-Plastic Analysis," **Journal of the Structural Division,** ASCE, Vol. 106, No. ST6, Proc. Paper 15501, June 1980.

28. Braestrup, M.W., and C.T. Morley, "Dome Effect in RC Slabs: Elastic-Plastic Analysis," **Journal of the Structural Division**, ASCE, Vol. 106, No. ST6, Proc. Paper 15502, June 1980.

29. Gabrielsen, B., C. Wilton, and K. Kaplan, **Response of Arching Walls and Debris from Interior Walls Caused by Blast Loading**, URS 7030-23, URS Research Company, San Mateo, CA, February 1975.

APPENDIX A
Examples of Upgraded Buildings

Appendix A
EXAMPLES OF UPGRADED BUILDINGS

INTRODUCTION

The purpose of this section is to provide examples of shelter upgrading plans as they would be applied in actual existing structures. The two levels of survival that were considered in these theoretical examples represent host area shelters, 2 psi, and key worker shelters, 50 psi, and the type, size, and spacing of the shoring systems were obtained, with some modifications, from the host area and key worker shelter manuals (Refs. A-1 and A-2).

The buildings used in these examples were selected from files of previous investigations conducted by SSI. The eight buildings used as examples of the upgrading for host area structures were obtained from SSI Report 8039-11 (Ref. A-3), and were all located in Rancho Mirage, California. The buildings selected include three from a hotel complex, and one each motel, market, market plaza, shopping center, and book store.

The two buildings used as examples of shelter upgrading in risk areas were both obtained from an underground school study conducted by SSI for the State of Oklahoma Civil Defense. One was the John Glenn Elementary School in Oklahoma City and the other the Wellston Elementary and Junior High School in Wellston, Oklahoma.

HOST AREA SHELTER UPGRADING PLANS

The three buildings from the Marriott Hotel complex and the California "6" Motel, Figures A-1 through A-4, were all ugpraded in a similar manner; i.e., one row of stud wall shoring at the centerline of each room using 2 in. by 4 in. wood studs

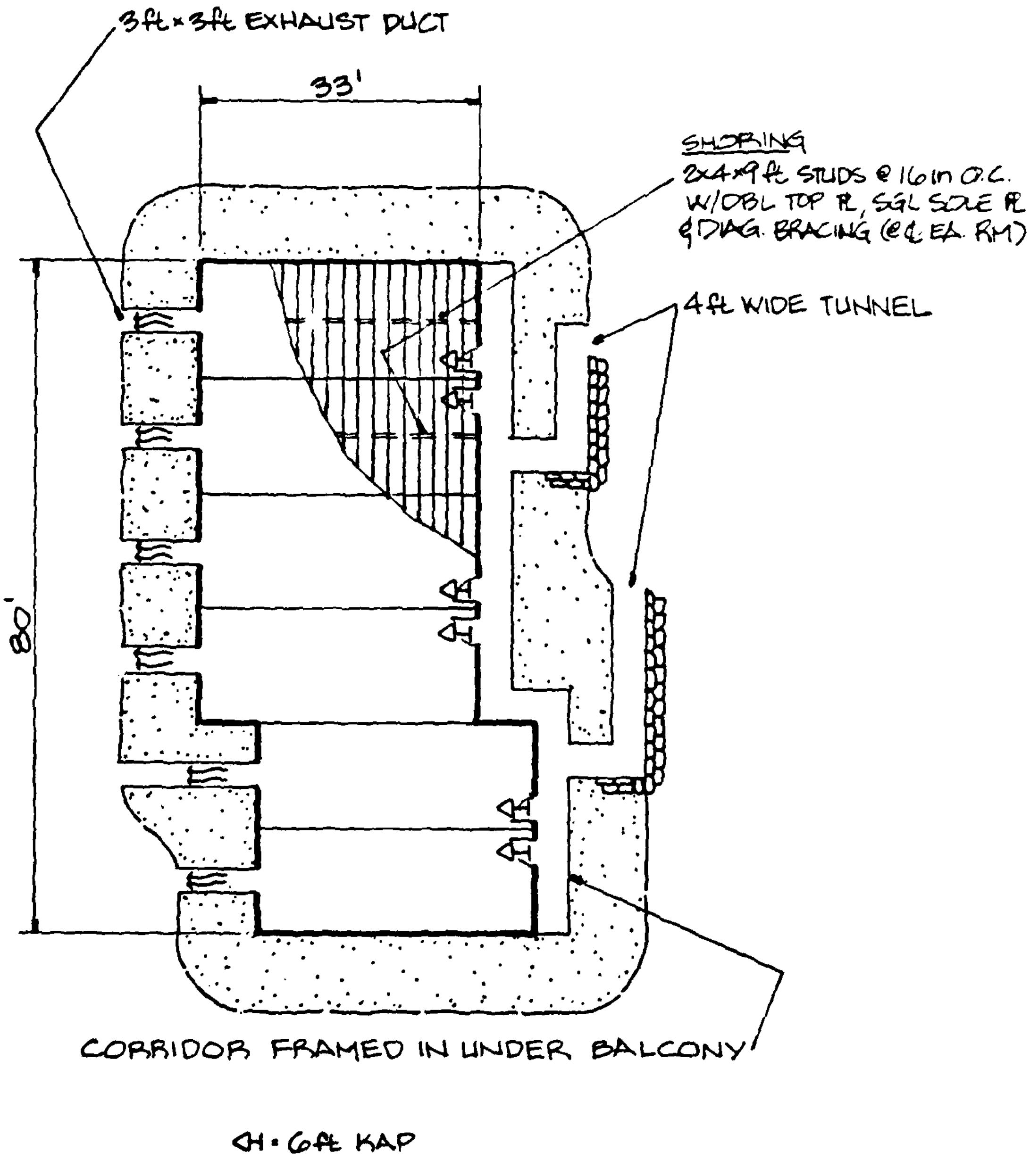


Fig. A-1. Upgrading Plan — Marriott Hotel, Small Unit.

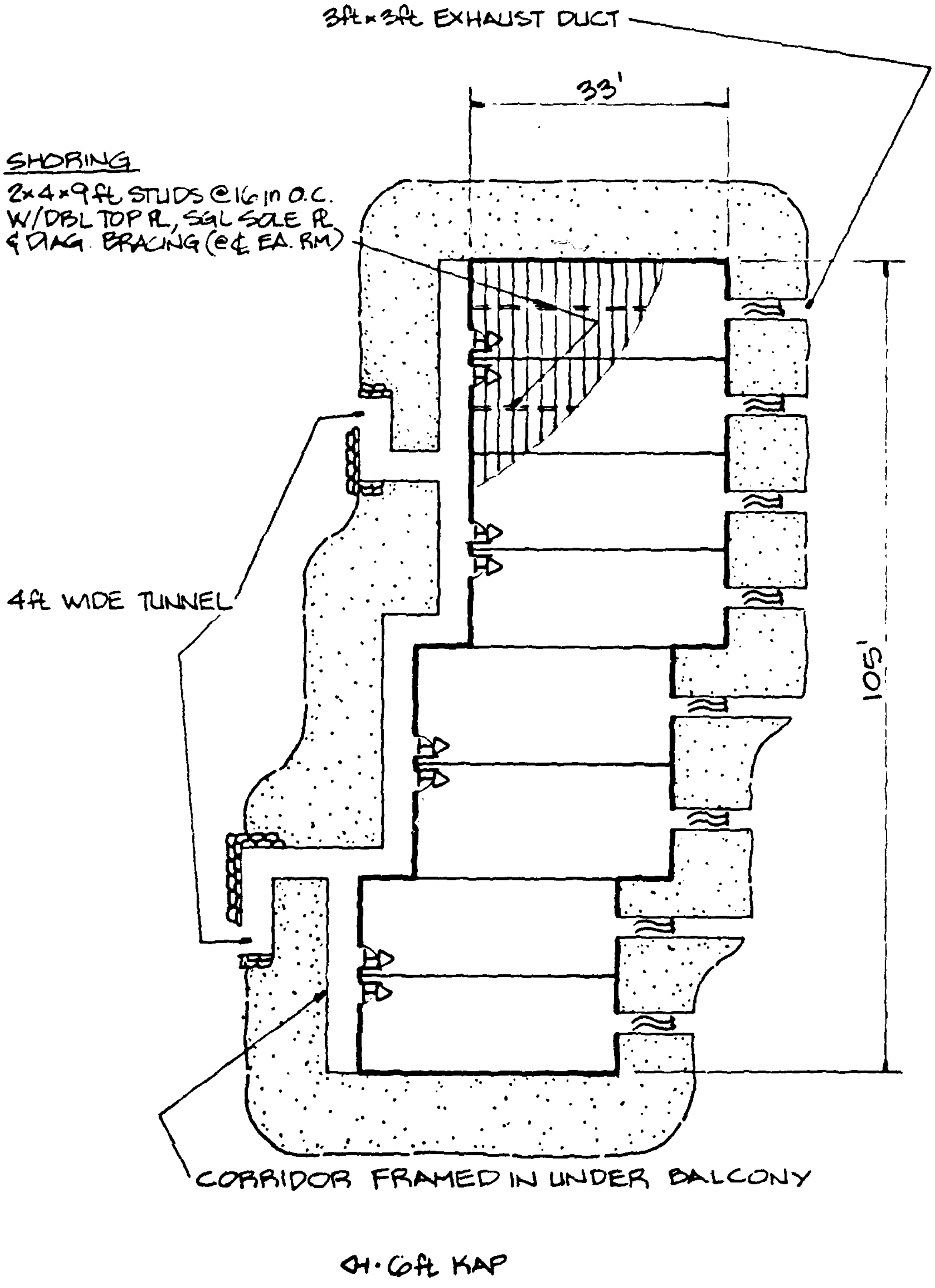


Fig. A-2. Upgrading Plan — Marriott Hotel, Medium Unit.

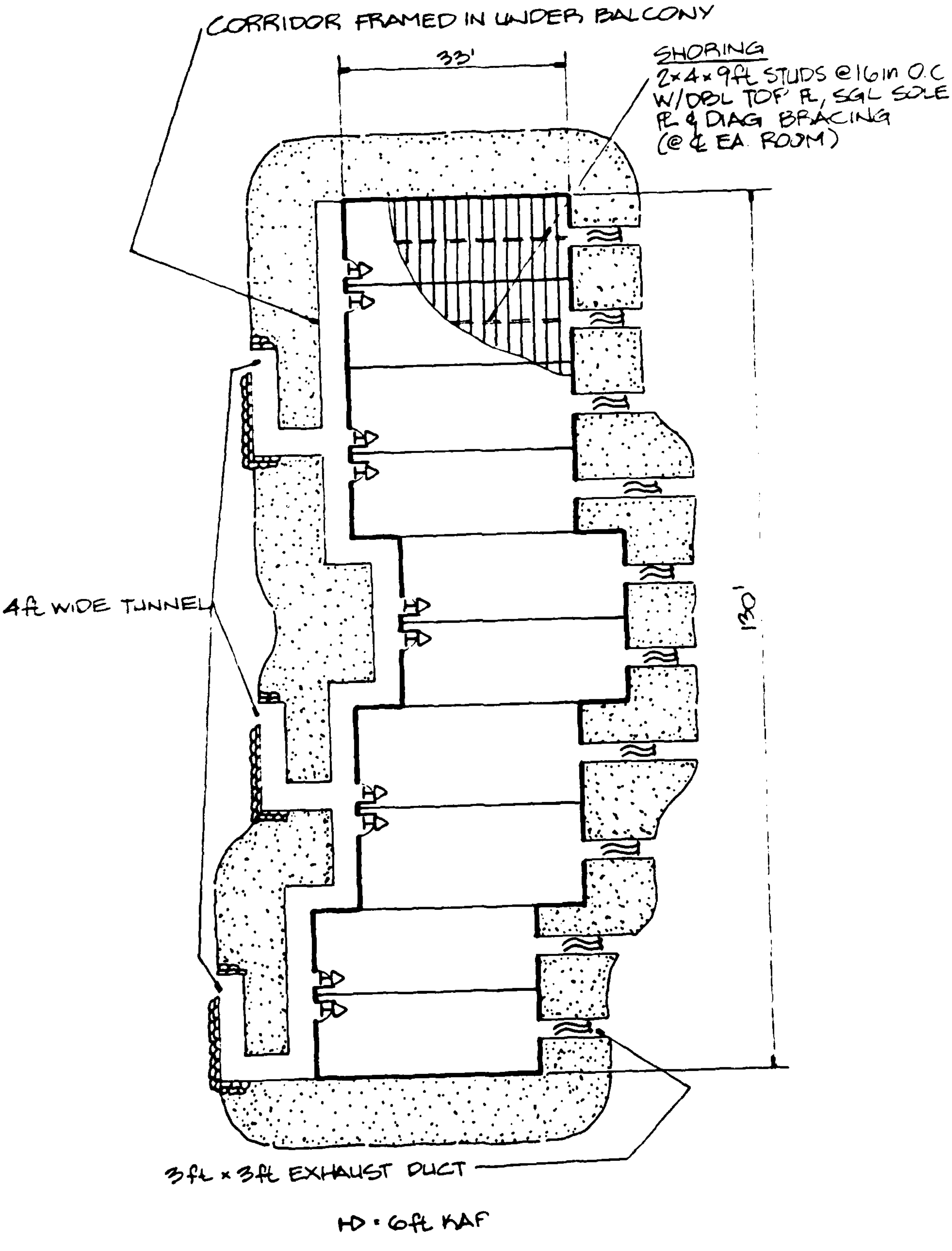


Fig. A-3. Upgrading Plan — Marriott Hotel, Large Unit.

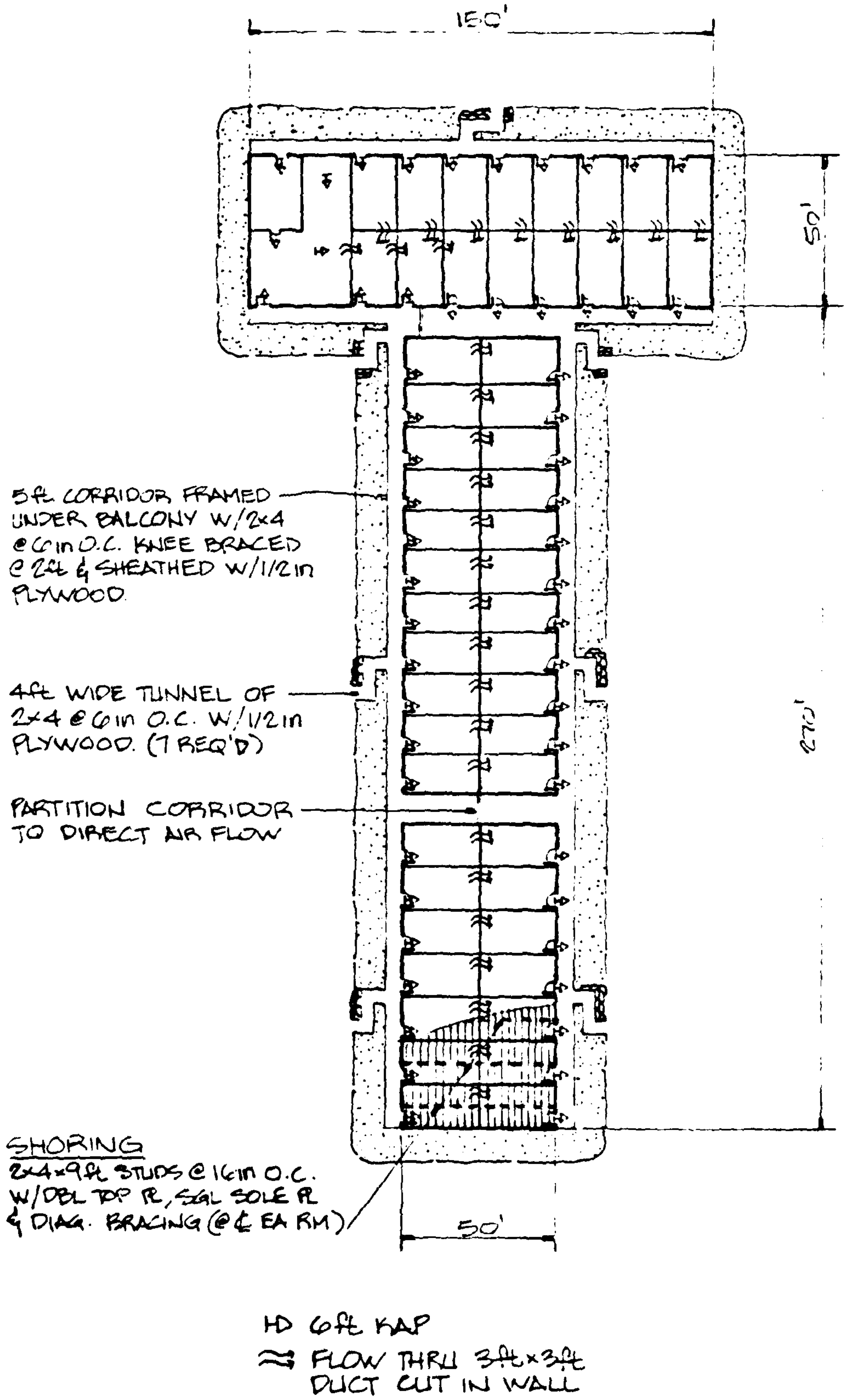


Fig. A-4. Upgrading Plan — California 6 Motel.

spaced 16 in. on center. Where present, the exterior corridors under the second floor balconies were framed in with wood studs, sheathed with plywood, and braced.

The Rancho Market, Figure A-5, was upgraded with post and beam shoring using one line of 6 in. by 6 in. timber posts, 8 ft on center, supporting 6 in. by 8 in. timber beams in each bay. The walls were upgraded with 4 in. by 4 in. timber at 12 in. on center.

The Rancho Market Plaza and the Rancho Las Palmas Shopping Center, Figures A-6 and A-7, were both shored with one line of wood stud wall shoring per room/bay. The shoring consisted of 2 in. by 4 in. studs, 24 in. on center. In both buildings the exterior corridors were framed in with wood studs and plywood sheathing, and braced.

The Bargain Books Store, Figure A-8, was shored with post and beam shoring that utilized either 6 in. by 6 in. timber beams supported by 4 in. by 4 in. timber posts at 6 ft on center, or 4 in. by 6 in. beams supported by 4 in. by 4 in. posts, 5 ft on center.

KEY WORKER AREA SHELTER UGRADING PLANS

Two existing underground school buildings used to develop the upgrading plans for risk area shelters are shown in Figures A-9 and A-10. The below ground portions of both schools were selected as the areas to be used for the upgrading examples.

Both of the structures are constructed of cast-in-place reinforced concrete. The roof of the Wellston School and the first floor of the John Glenn School are both one-way slab construction. Reinforced concrete spread footings support both the exterior and interior walls, and reinforced concrete slab on grade construction is used for the floor slabs. The roof slab in the Wellston School is 11 in. thick, and the first floor slab in the John Glenn School is 10½ in. thick.

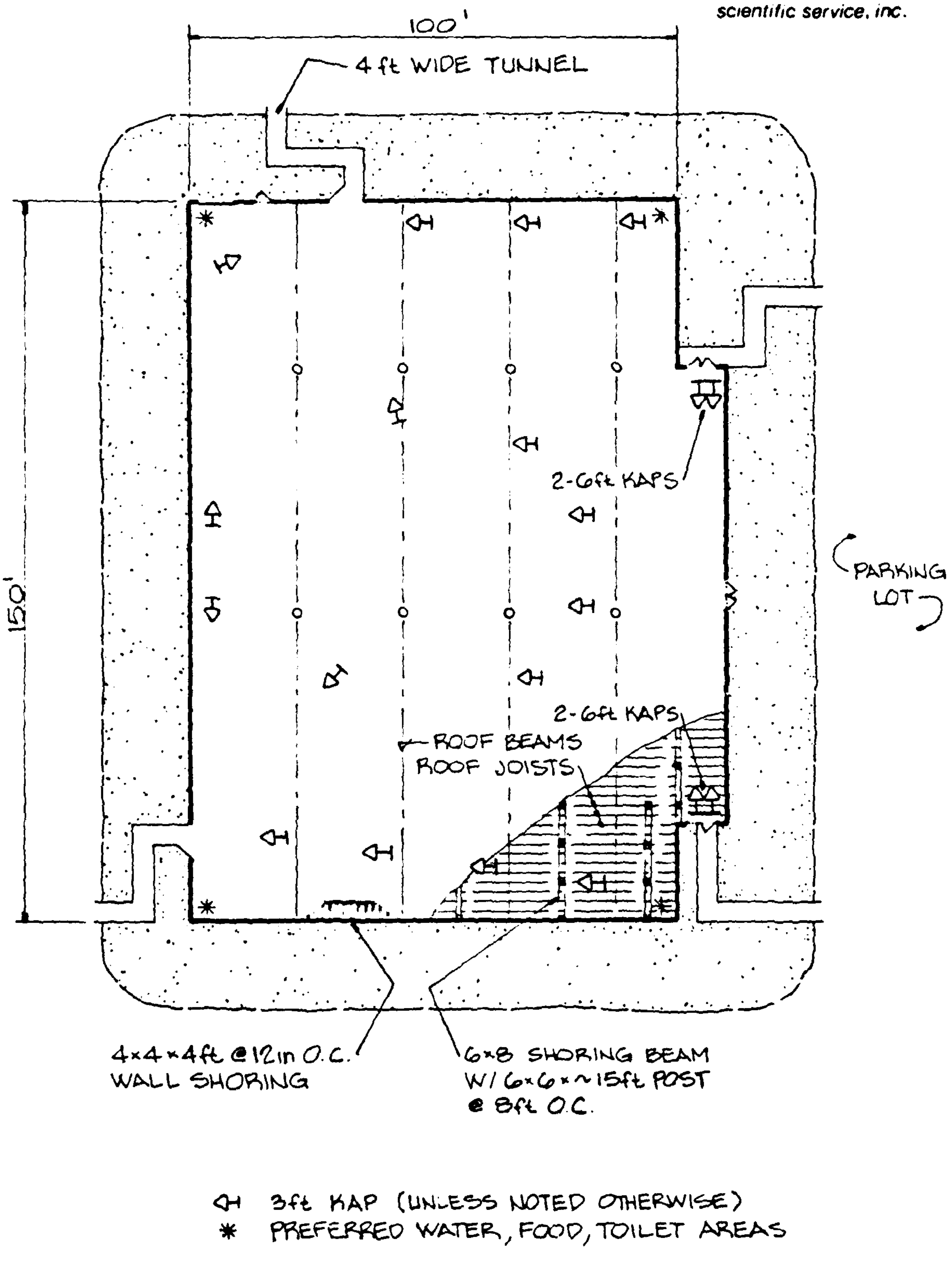
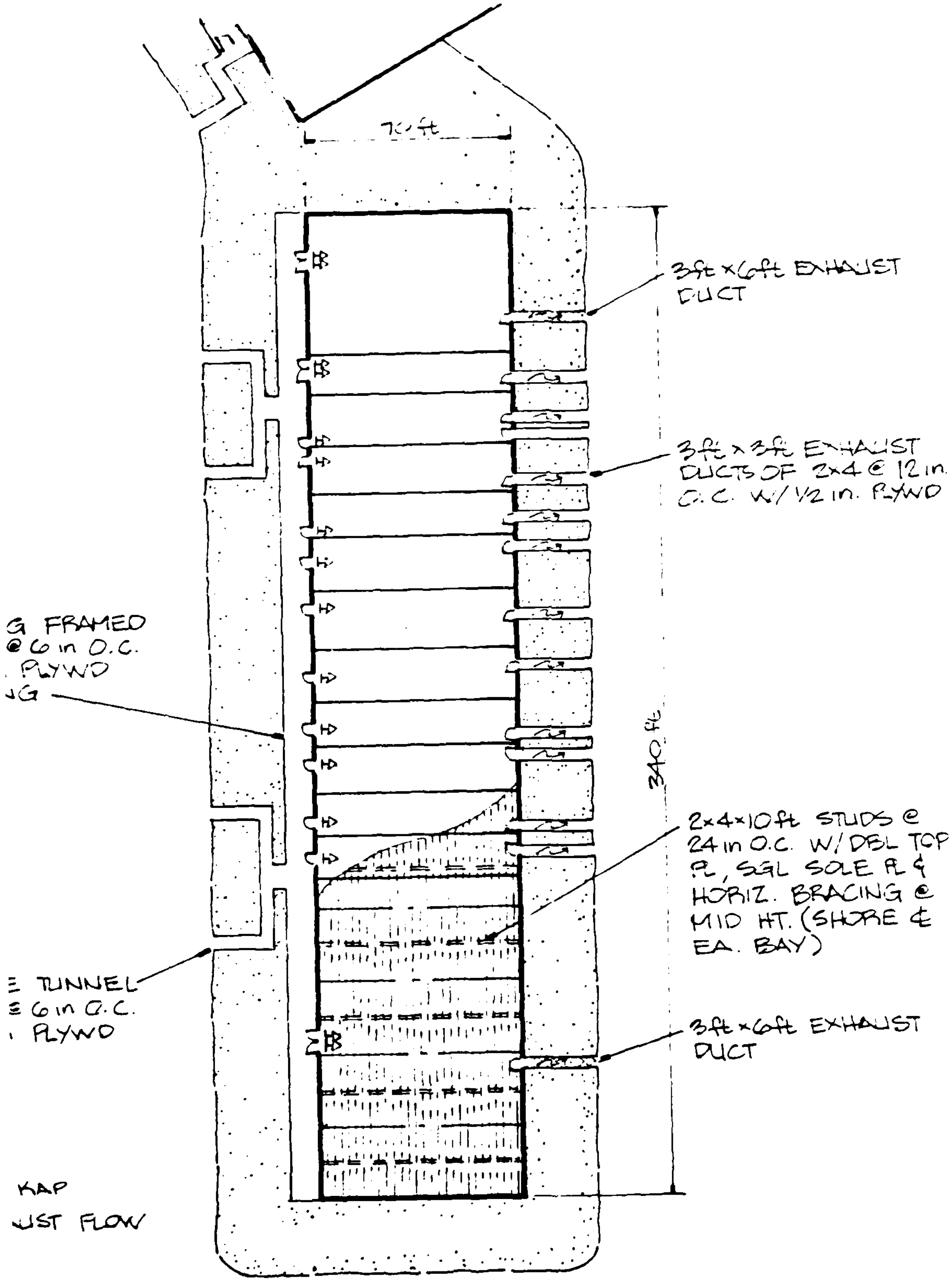


Fig. A-5. Upgrading Plan — Rancho Market.



. A-6. Upgrading Plan — Rancho Market Plaza.

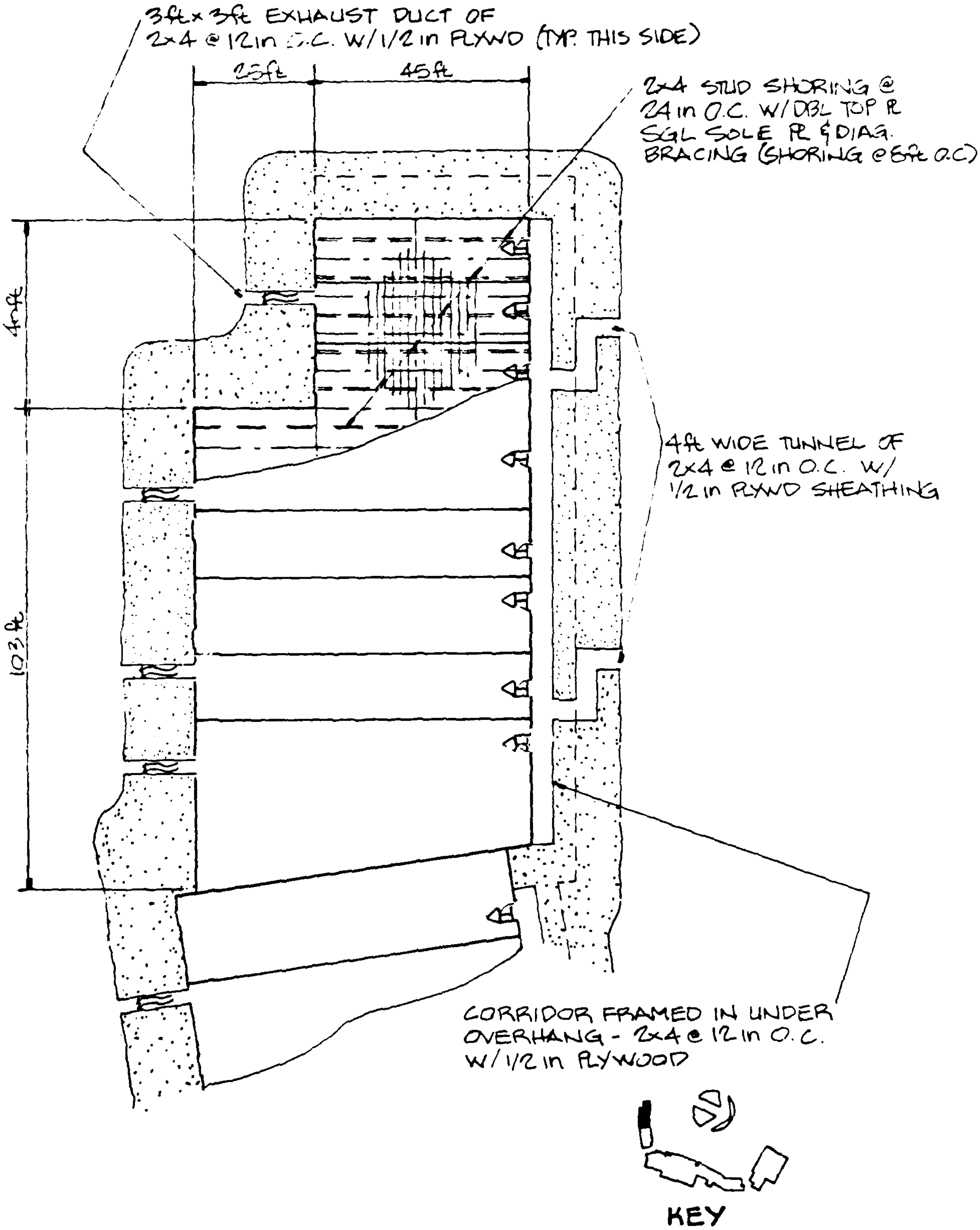


Fig. A-7. Partial Upgrading Plan — Rancho Las Palmas Shopping Center.

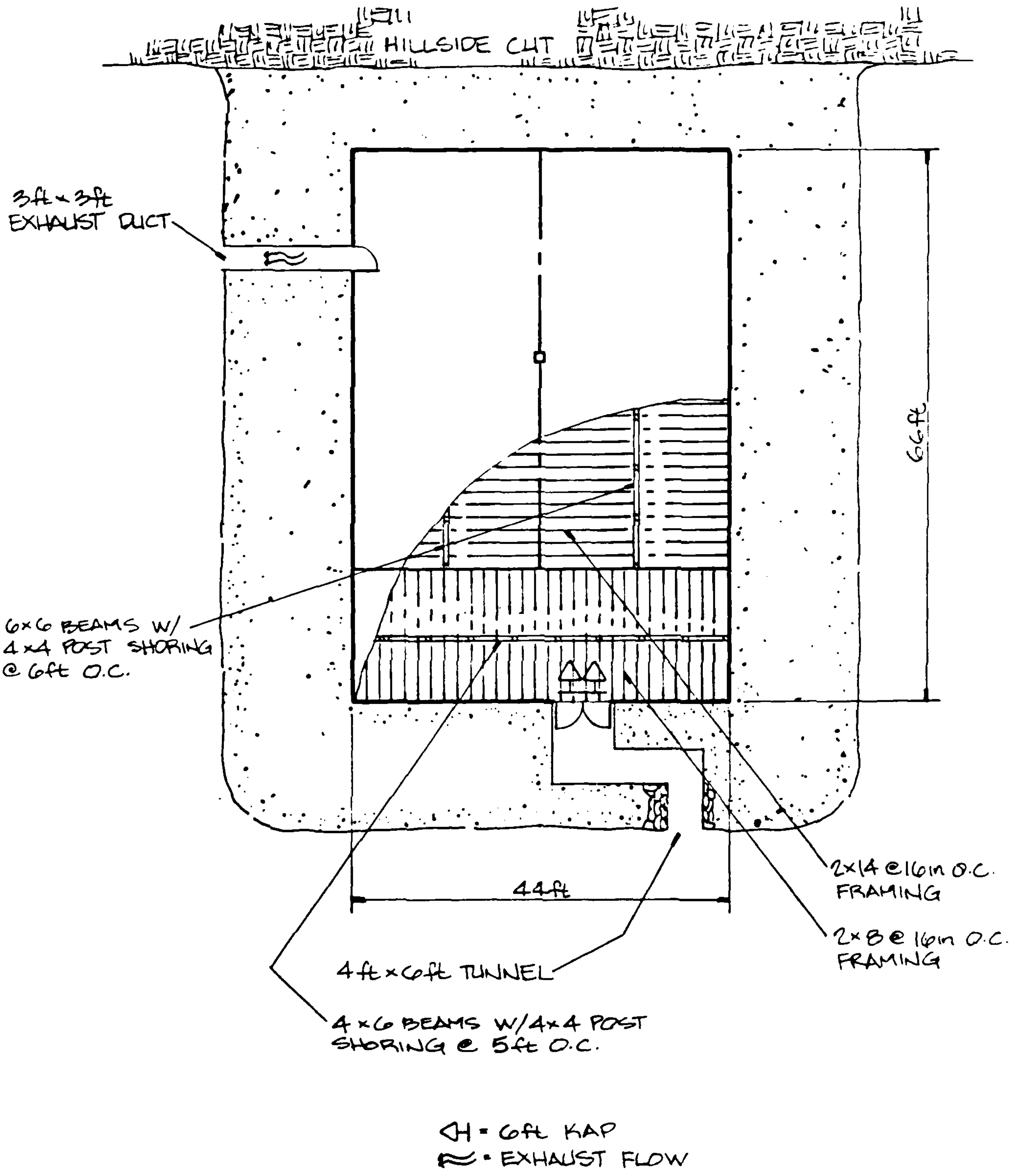
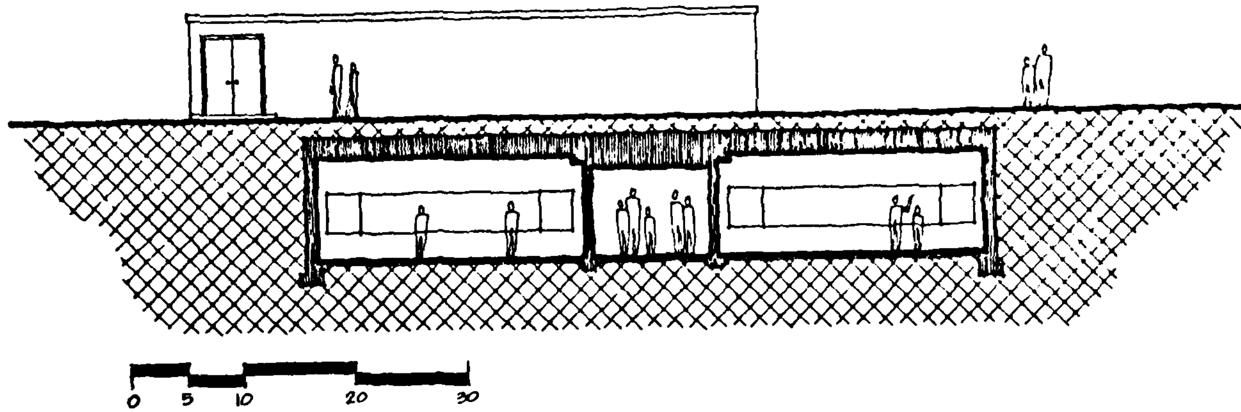
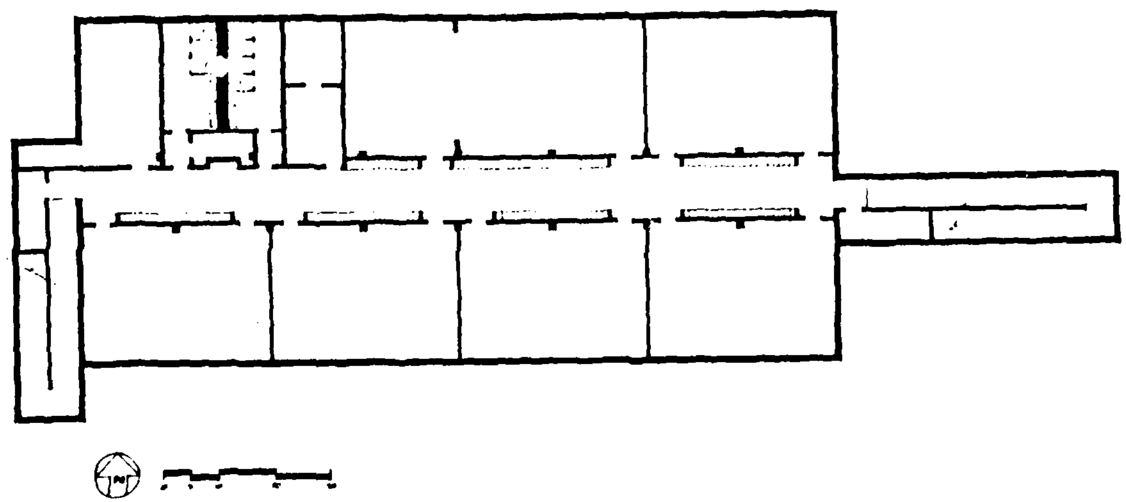


Fig. A-8. Upgrading Plan — Bargain Books Store.

WELLSTON

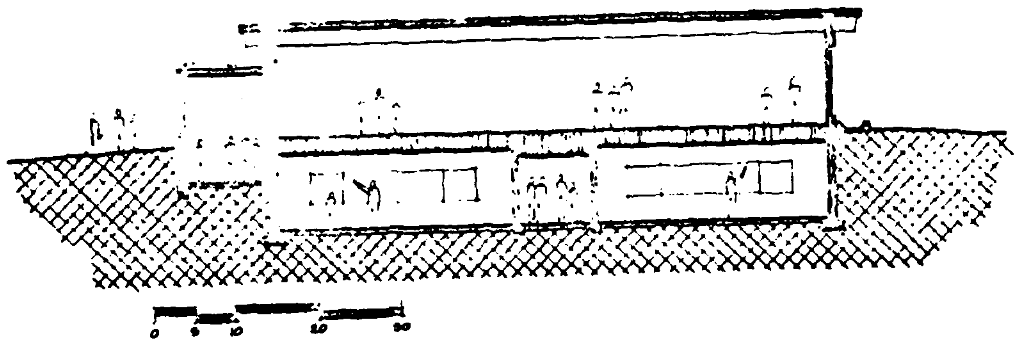


Section



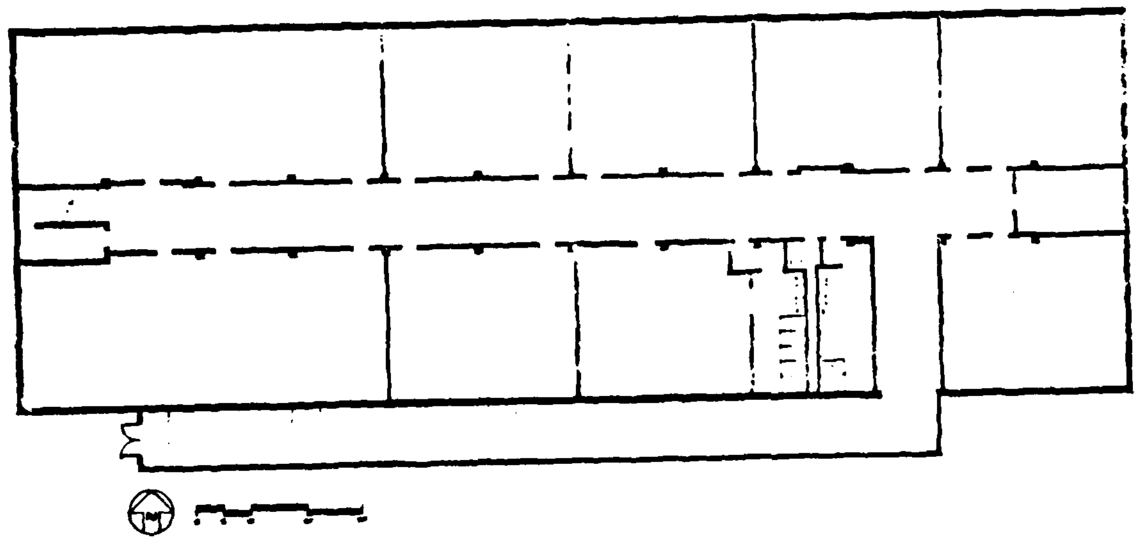
Floor Plan

Fig. A-9. Plan of Wellston, Oklahoma, School.



Section

JOHN GLENN



Floor Plan

Fig. A-10. Plan of John Glenn School, Oklahoma City.

Fortunately, a considerable amount of the basic design information was available in our files on both of these buildings, including the sizes and location of the reinforcing steel. As a result of these available data, it was possible to conduct a reasonably rigorous analysis on both of these slabs. This type of analysis results in a higher degree of accuracy in the prediction of the collapse loads than might be attained using the key worker shelter manual (Ref. A-2) where, by necessity, the prediction methodology was developed on a general and conservative basis.

While both structures have essentially the same type of construction and practically the same roof spans, the concrete reinforcement in the two buildings is different, especially in the hallways. The result is that the John Glenn School with significantly more reinforcement in the hallway requires only a single row of shoring, while the Wellston School requires two rows of shoring.

The overall shoring plan for the Wellston School is shown on Figure A-11 and a half-section through the shored area is shown on Figure A-12. Post and beam shoring is used throughout. Shoring in the classrooms is at the quarter points using steel wide flange beams atop 8 in. by 10 in. wood posts. The hallway shoring consists of two rows of 6 in. by 8 in. wood posts, which support steel wide flange beams.

The shoring plan for the John Glenn School is shown in Figure A-13 and a section through the shored area in Figure A-14. Post and beam shoring is used once again throughout the building. Shoring in the classrooms is at the quarter points using steel wide flange beams supported atop 10 in. by 10 in. wood posts. Hallway shoring consists of a single row of post and beam shoring at midspan using 10 in. by 10 in. wood posts and steel wide flange beams.

The analysis conducted on these slabs indicated that, with the shoring sizes and configuration as described, the survival overpressures for the Wellston School would be 50 psi, and for the John Glenn School, 67 psi.

It will noted that these key worker shelter upgrading designs do not include entranceway enclosures. The primary reason for this is that there is none that we

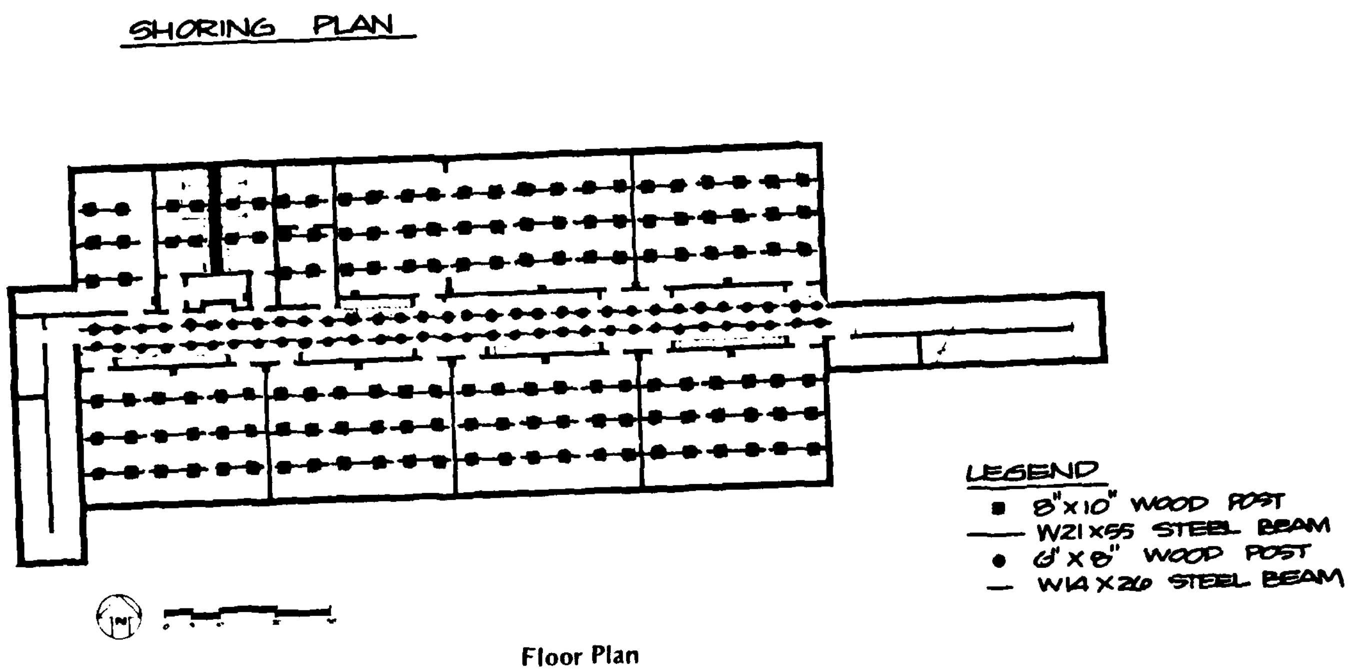


Fig. A-11. Shoring Plan — Wellston School.

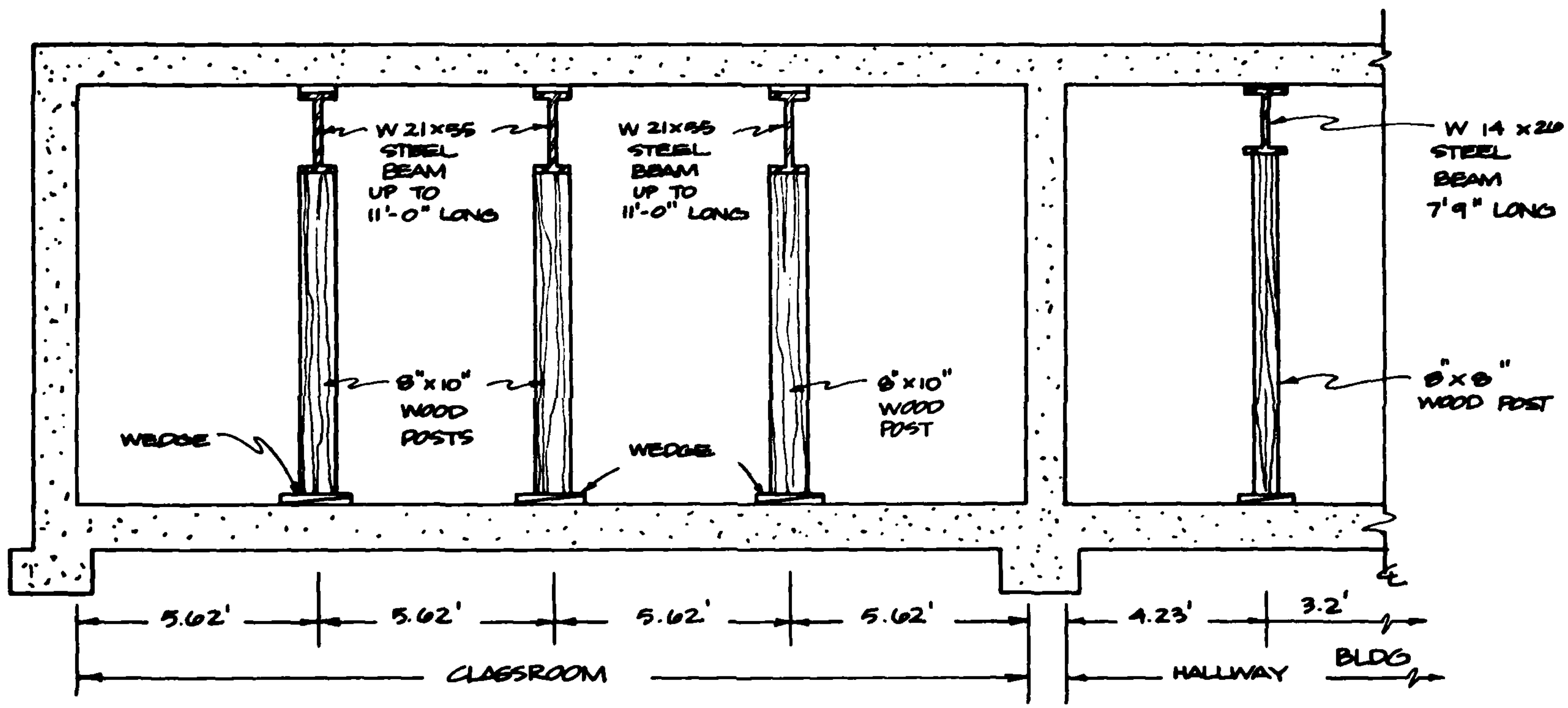


Fig. A-12. Shoring of the Wellston Elementary School.

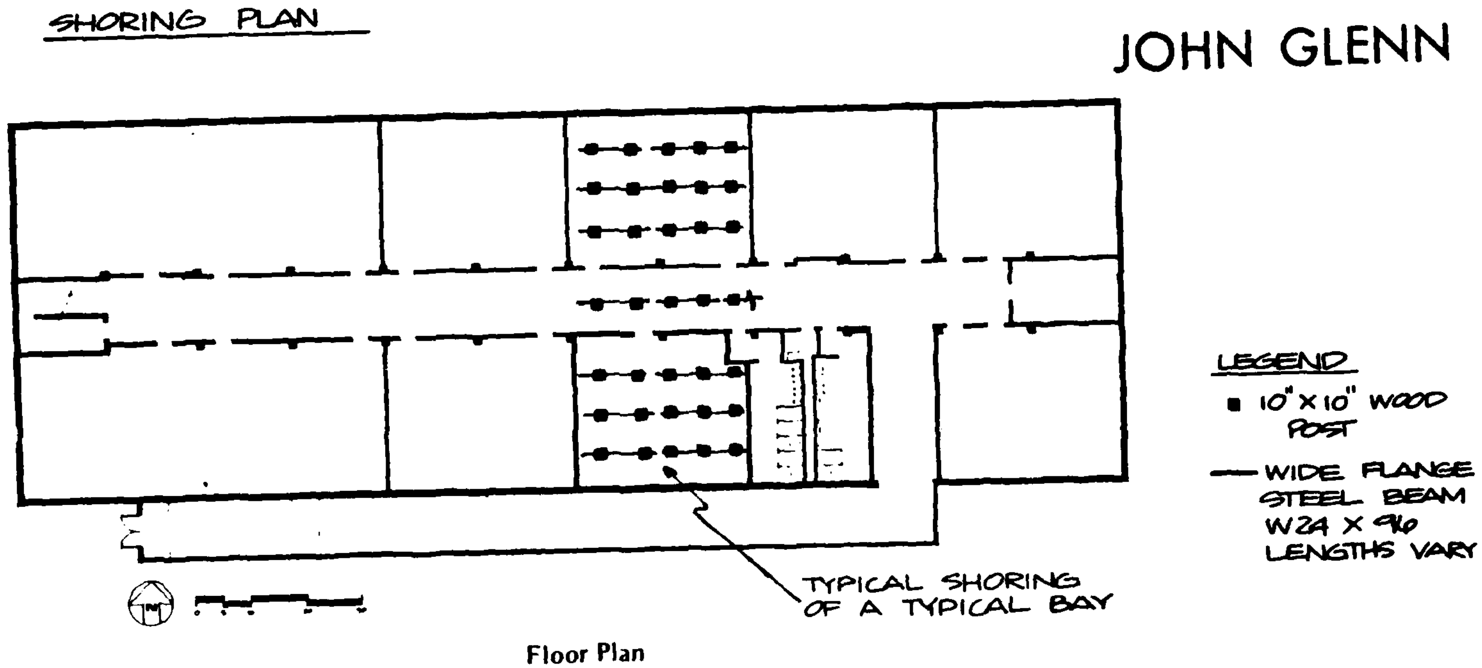


Fig. A-13. Shoring Plan — John Glenn School.

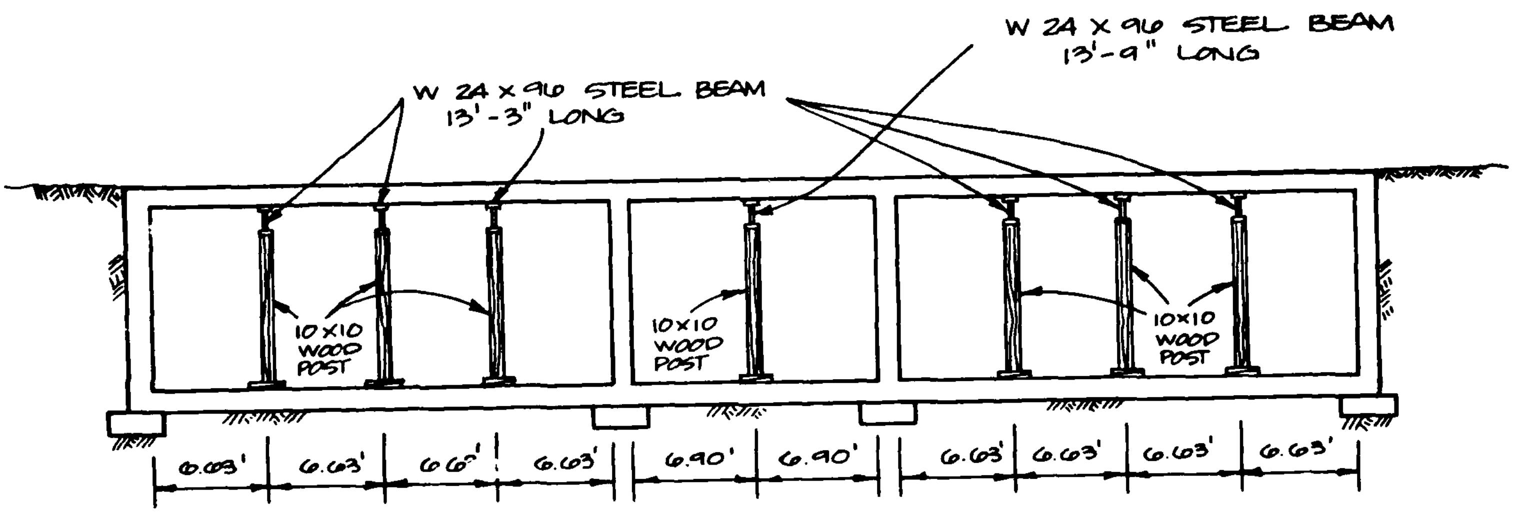


Fig. A-14. Shoring of the John Glenn Elementary School.

know of that is sufficiently proof-tested to the 95% probability of survival level, the criterion used for all upgrading designs. Some candidates exist, however, including an arched steel door tested by the Waterways Experiment Station, a massive concrete sliding door tested at 50 psi using a 37 kt weapon during Operation Plumbbob (Ref. A-4), steel grates currently being tested at the Ballistic Research Laboratory, and a few prototype reinforced concrete and steel designs developed by SSI that have not been tested.

Various types of concrete and steel culverts have been tested, and these could be placed in the entranceways and backfilled with soil and the holes covered with plank or steel plate doors. The required thickness of these doors can be determined using the guidance included in the Key Worker Shelter Manual (Ref. A-2). These 2 ft to 3 ft diameter pipes, however, do not make adequate entranceways. They are difficult to travel through and usually are not adequate for ventilation purposes.

REFERENCES

A-1 Wilton, C., B.L. Gabrielsen, and R.S. Tansley, **Shelter Upgrading Manual: Host Area Shelters**, SSI 7815-8, Scientific Service, Inc., Redwood City, CA March 1980.

A-2 Tansley, R.S., and R.D. Bernard, **Shelter Upgrading Manual: Key Worker Shelters**, SSI 8012-7, Scientific Service, Inc. Redwood City, CA May 1981.

A-3 **Prototype Crisis Shelter Development Plans in Host Areas,** SSI 8039-11, draft final report, Scientific Service, Inc., Redwood City, CA, July 1981.

A-4 **CEX-68.3,** Nuclear Weapons Effects Tests of Blast Type Shelters, U.S. Atomic Energy Commission, June 1969.

APPENDIX B

Prediction Methodology

Appendix B
PREDICTION METHODOLOGY

INTRODUCTION

The effort during the first year has been devoted to gathering and analyzing data to be included in the various indices of the prediction model, particularly the building index. The data analyzed to date have been obtained from a number of sources, including the extensive laboratory testing program conducted by SSI over the last five years, published test data by others, and reports issued by private firms and governmental agencies on the results of investigations of actual building failures that occur periodically.

The data gathering and analysis will continue throughout the second year; however, the primary emphasis will now be directed at developing well-defined lists of the building element types, ranked and weighted as to their individual performance under blast loading. This is the necessary second step prior to combining these elements into complete structures for ultimate evaluation and rating. The SSI testing program will continue into the second year, and as before, will provide the bulk of the required data. Other sources of data will be utilized as published when appropriate. The methodology that will be used in developing the ranking and weighting of the various structural elements is discussed in more detail below.

BUILDING ELEMENTS

It is intended that the building index will take into account each of the primary structural elements that make up a building, such as the floors, roofs, and walls. The connections that occur between these elements, although not considered structural elements as such, will also be addressed early in the index development

for reasons that will be discussed later. The building index will be the foundation of the prediction model, and will require development in a manner that will permit the planner to rate and select the best building with a minimal amount of effort. The methodologies that we intend to use in developing the ranking procedure for each of these primary structural elements are outlined below.

Floor Systems

The rating of each type of floor system will follow the general evaluation and selection process developed for use in the shelter upgrading manuals (Refs. B-1 and B-2), and will be based basically on the "intended use" of the floor. With few exceptions, most buildings constructed during the past 50 years were designed using some type of building code. These codes recommend the minimum design loads for floors for each category of occupancy. A typical listing of these recommended loads is shown in Table B-1.

The code used for a particular building design may have been one of the three or four national codes, or the design may have been based on a local code, which is usually an adaptation of one of the national codes with only minor revisions. The particular code that was used is not a problem with respect to the rating of floor systems, since the recommended load requirements are quite similar in all codes, and the various occupancy categories can be conveniently grouped by loads.

A second factor that will enter into the rating system will be the type of construction. A wood floor, for example, designed for a "light storage" occupancy, 125 psf (see Table B-1), would not necessarily have the same collapse load as a concrete floor designed for an identical occupancy. The reason for this is inherent in the design methods used and the manufacture and/or selection of the construction materials; i.e., different design methods are required for different materials, many incorporating different parameters and saftey factors.

Roof Systems

The approach to rating the various types of roof systems will be somewhat different from that used for floors. The basic minimum design load for roofs, as recommended in the building codes, is usually a relatively constant value of 20 psf

TABLE B-1: DESIGN INFORMATION — RECOMMENDED MINIMUM FLOOR LIVE LOADS

UNIFORMLY DISTRIBUTED LOADS

Occupancy or Use	Live Load (psf)
Apartments (see Residential)	
Armories and drill rooms	150
Assembly halls and other places of assembly:	
Fixed seats	60
Movable seats	100
Platforms (assembly)	100
Bowling alleys, poolrooms, and similar recreational areas	75
Corridors:	
First floor	100
Other floors, same as occupancy served except as indicated	
Dance halls and ballrooms	100
Dining rooms and restaurants	100
Dwellings (see Residential)	
Garages (passenger cars only)	50
Grandstands (see Reviewing stands)	
Gymnasiums, main floors and balconies	100
Hospitals:	
Operating rooms, laboratories	60
Private rooms	40
Wards	40
Corridors, above first floor	80
Hotels (see Residential)	
Libraries:	
Reading rooms	60
Stack rooms (books & shelving at 65 pcf) but not less than	150
Corridors, above first floor	80
Manufacturing:	
Light	125
Heavy	250
Office buildings:	
Offices	50
Lobbies	100
Corridors, above first floor	80
File and computer rooms require heavier loads based upon anticipated occupancy	
Penal institutions:	
Cell blocks	40
Corridors	100
Residential:	
Multifamily houses:	
Private apartments	40
Public rooms	100
Corridors	80
Dwellings:	
First floor	40
Second floor and habitable attics	30
Uninhabitable attics	20
Hotels:	
Guest rooms	40
Public rooms	100
Corridors serving public rooms	100
Corridors	80
Reviewing stands and bleachers	100
Schools:	
Classrooms	40
Corridors	80
Sidewalks, vehicular driveways, and yards, subject to trucking	250
Skating rinks	100
Stairs and exitways	100
Storage warehouse:	
Light	125
Heavy	250
Stores:	
Retail:	
First floor, rooms	100
Upper floors	75
Wholesale	125
Theaters:	
Aisles, corridors, and lobbies	100
Orchestra floors	60
Balconies	60
Stage floors	150
Yards and terraces, pedestrians	100

DESIGN LOAD GROUPS

(as used in this manual)

LIGHT: *40 to 60 psf*

MEDIUM: *80 to 125 psf*

HEAVY: *150 to 250 psf*

This value is permitted to be reduced in increments to a maximum of 40%, or 12 psf, as the slope of the roof increases, with the first incremental reduction permitted when the roof slope exceeds 4 in. per foot. Since a sloping roof surface greater than 4 in. per foot presents difficulties with respect to the placement of radiation protection, and since this load reduction is judged not to be a significant factor in rating roofs for the intended purpose of the prediction model, all roofs are to be rated based on a conservative design load value of 20 psf. Additionally, application of this permitted reduction in roof design loading would only greatly complicate the planner's shelter selection process without any tangible increase in shelter spaces.

An important factor that will be included in the roof rating system is the increased load capability built into roof structures in many areas of the country because of snow loading. The snow load, when added to the 20 psf minimun design load mentioned above, often increases the required design of roof systems in excess of 30 psf to 40 psf, depending on the area of the country. Figure B-1 shows a typical snow load map of the United States used for design purposes. As may be seen on this map, many roof systems in snow regions may be designed for loading that actually exceeds some floor loadings. This condition is prevalent enough to warrant incorporation of these data into the roof system rating index.

There are many areas of the country, particularly in mountainous regions,where the recommended snow loads are related to topography and vary considerably, sometimes by several hundred pounds per square foot within 10 to 20 miles. These areas will require a special rating system that will group these various design snow loads into rather large and firmly defined geographical areas, so that the planner may efficiently and conservatively rate the buildings with minimal effort.

Wall Systems

A considerable amount of previous work has been performed on the blast resistance of wall systems without backfill (Ref. B-3), and current investigations under this contract are addressing the resistance of basement walls and walls above grade that are bermed or backfilled. Based on the data available to date, the rating methodology that we intend to use in this year's effort to develop a wall rating index for non-bermed above grade walls is to be based primarily on the mass of the wall.

Fig. B-1. Snow Load in Pound-Force per Square Foot on the Ground, 50-Year Mean Recurrence Interval.

A review of the data from Ref. B-3 indicates some correlation between the mass of the wall and blast resistance, and accordingly, these data will be assimilated and analyzed. Should this mass-related rating method be substantiated, this approach will be used to promulgate this particular wall system rating index.

The rating index for basement and bermed walls initially will be very preliminary because of the lack of viable data in these areas. It is anticipated that the investigative programs conducted and/or completed under this contract during the next several years will provide substantial input toward the development of this index. Included in the next year's effort are full-scale tests of bermed walls. These tests are directed at defining the effects that the slope of berm and the degree of compaction of the bermed soil will on the resultant loading to the wall. Additionally, the completion this year of the testing and analysis of the performance of the basement test walls at the MILL RACE event, supplemented by tests of 1/20th scale models of similar walls in the 12-in. shock tube, is anticipated. As data are developed and become available from both of these ongoing investigations, they will be appropriately incorporated into the index.

Connections

As stated previously, it is now our intent to include only the three basic structural elements, floors, roofs, and walls, in the building index. However, as the test and analytical programs progressed through the first year, it became apparent that the performance of many types of primary structural elements was directly related to the performance of their associated connections. In several cases, the full-scale tests indicated that the integrity of the connections controlled the load-carrying capabilities of the structural elements. Also, we have found that many times it is cost effective to conduct the connection tests associated with a particular structural element in conjunction with the full-scale tests on that element.

During the first year's effort, considerable data were developed with respect to structural connections, and it is anticipated that these data will be substantially increased during the second year. Included in the first year's program was a comprehensive survey and analysis of the expected performance under blast loading

of concrete structural connections. It is the intent to continue this analysis into the second year, including both steel and/or timber connections. Also included in the second year's planning are tests on selected connections.

It is realized that the ranking index to be developed for connections will, in all probability, only be used to modify the primary building index. However, because of the importance of connections to the overall prediction model, it is expedient to address structural connections early in the formation of the model, and to begin to rank connection performance as the data become available.

CONCLUSIONS

When the indexing of the primary structural elements is completed, and simplified to the maximum extent possible, we believe that it will provide the initial basis for the development of the complete methodology for comparative selection of the best shelter spaces available. With minimum effort, planners who may only have a cursory knowledge of building design and construction, will be in a position to make the required comparisons.

REFERENCES

B-1 Wilton, C., B.L. Gabrielsen, and R.S. Tansley, **Shelter Upgrading Manual: Host Area Shelters,** SSI 7815-8, Scientific Service, Inc., Redwood City, CA, March 1980.

B-2 Tansley, R.S., and R.D. Bernard, **Shelter Upgrading Manual: Key Worker Shelters**, SSI 8012-7, Scientific Service, Inc., Redwood City, CA, May 1981.

B-3 Wilton, C., K. Kaplan, and B.L. Gabrielsen, **The Shock Tunnel: History and Results**, SSI 7618-1, Scientific Service, Inc., Redwood City, CA, March 1978.

Appendix C
CALCULATIONS FOR GLULAM BEAM TESTS

This section of the report contains the calculations used to determine the design load capacity of the glulam beam tests conducted and reported in Section 3. Two sets of calculations are included, one for the 6-3/4 in. by 16-1/2 in. beams designed with typical floor loading, 50 psf, and one for the 3-1/8 in. by 18 in. beams designed with typical roof loading, 20 psf. The test assemblies were designed in accordance with the codes and standards applicable to these types of members, as listed below:

1979 Edition of the Uniform Building Code (Ref. C-1)

AITC Timber Construction Manual (Ref. C-2)

Western Wood Products Association Product Use Manual (Ref. C-3)

1977 National Design Specification for Wood Construction (Ref. C-4)

1978 Edition of Glulam Systems by AITC (Ref. C-5)

April 1978 Revision of Plywood Design Specification by the American Plywood Association (Ref. C-6)

KC Metals 1981 Handbook of Structural Design & Load Values (Ref. C-7)

The following calculations indicate the design methods used for all of the test assemblies. The notation used is defined in Table C-1.

TABLE C-1: NOTATION

A = Cross-sectional area

C = Plywood constant equal to 60 for panels with face grain parallel to supports

C_d = Depth correction factor

D = Dead load per square foot of area supported by the member

E = Modulus of elasticity

F_b = Allowable bending stress

F_s = Allowable shear, rolling, in the plane of plies

F_v = Allowable shear stress

f_b = Actual bending stress

f_v = Actual shear stress

I = Moment of inertia

Ib/Q = Plywood rolling shear constant

KS = Plywood effective section modulus

LL = Live load

L = Unit live load per square foot of area supported by the member

ℓ = Span, center to center bearing

ℓ_1 = Span, center to center of supports

ℓ_2 = Clear span, center to center span minus support width

ℓ_3 = Clear span plus support width factor (0.25 in. for 2-in. nominal framing)

M = Actual moment

R = Support reaction, in lb

S = Section modulus

$T.A$ = Tributary area

t = Nominal plywood panel thickness

TABLE C-1: NOTATION (contd)

V = Actual shear

W = Total load per foot

W_D = Dead load per foot

W_L = Live load per foot

Δ_{ALLOW} = Allowable deflection

Δ_{D+L} = Deflection as a result of dead + live load

Δ_b = Plywood bending deflection

Δ_L = Deflection as a result of live load

Δ_s = Plywood shear deflection

FLOOR SYSTEM DESIGN CALCULATIONS

LOADS

FLOOR LOADS

	DEAD LOADS		LIVE LOADS
PLY SHEATHING	3.0 PSF		
UNDERLAYMENT	3.0 PSF		50 PSF
2×8's @ 24" O.C.	1.4 PSF		
MISC'H	4.0 PSF		
	11.4 PSF	← TO SUB PURLINS	
4"×12" @ 8'0" O.C.	1.2 PSF		
	12.6 PSF	← TO PURLINS	
6¾"×10½" @ 12'0" O.C.	2.3 PSF		
	14.9 PSF	← TO GLULAMS & POSTS	

SUB-PURLINS

SPAN 8' T.A. = 16 FT^2

$W_D = 2(11.4) = 22.8$ PLF

$W_L = 2(50) = 100$ PLF

125 PLF

$A = 10.88$ IN^2

$S = 13.14$ IN^3

$I = 47.63$ IN^4

TRY 2×8's

$F_b = 1450$ PSI

$F_v = 95$ PSI

$E = 1.7 \times 10^6$ PSI

$$M = \frac{W\ell^2}{8} = \frac{123(8)(96)}{8} = 11808 \text{ LB-IN}$$

$$V = \frac{W\ell}{2} = \frac{123(8)}{2} = 492 \text{ LB}$$

$$f_b = \frac{M}{S} = \frac{11808}{13.14} = 899 \text{ PSI} < 1450 \text{ PSI}$$

$$f_v = \frac{1.5V}{A} = \frac{1.5(492)}{10.88} = 67 < 95$$

$$\Delta_{D+L} = \frac{5(123)(8)(96)^3}{384(1.7\times10^6)(47.6)} = 0.14" = \frac{\ell}{685} < \frac{\ell}{240}$$

$$\Delta_L = 0.14\left(\frac{100}{123}\right) = 0.114" = \frac{\ell}{843} < \frac{\ell}{360}$$

USE 2"×8" D.F. No. 2
@ 2'-0" O.C.

PURLIN

SPAN 12'0" T.A. = 96 FT^2

$W_D = 12.6(8') = 101.$ PLF

$W_L = 50(8) = 400$ PLF

$W = 501$ PLF

$A = 39.38$ IN^2

$S = 73.83$ IN^3

$I = 415.3$ IN^4

TRY A 4"×12"

$F_b = 1500$ PSI

$F_v = 95$ PSI

$E = 1.8 \times 10^6$ PSI

$$M = \frac{501(12)(144)}{8} = 108200 \text{ LB-IN}$$

$$V = \frac{501(12)}{2} = 3006 \text{ LB}$$

$$f_b = \frac{108200}{73.8} = 1466 < 1500 \text{ PSI}$$

$$f_v = \frac{1.5(3006)}{39.4} = 114$$ O.K. IF NO END CHECKS ALLOWED

$$\Delta_{D+L} = \frac{5(501)(12)(144)^3}{384(1.8\times10^6)(415.3)} = 0.312"$$

$$\Delta_{D+L} = 0.312" = \frac{\ell}{461} < \frac{\ell}{360}$$

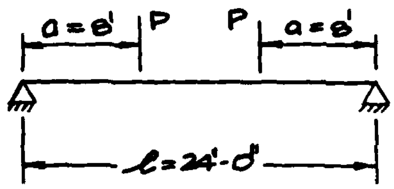
$$\Delta_L = \frac{400}{501} \times 0.312 = 0.249 = \frac{\ell}{578} < \frac{\ell}{240}$$

USE 4"×12" D.F. No. 1 @ 8'-0" O.C.

GLULAM SPAN 24'-0" T.A. = 288 FT^2

D.L. = 14.9 PSF
L.L. = 50 PSF

W_D = 12' (14.9) = 179 PLF
W_L = 12' (50) = 600 PLF
779 PLF



a = 8'-0"
ℓ = 24'-0"
P = W8' = 779(8) = 6232 LB

TRY A 6¾" × 10½" 24 F
A = 111.4 IN^2 $*F_b' = 2253$ PSI
S = 306.3 IN^3 $F_v = 165$ PSI
I = 2527 IN^4 $E = 1.8 \times 10^6$ PSI
$C_d = 0.97$ $F_{c\perp} = 450$ PSI

REDUCTION FOR A 2 FT LOAD = 0.968

$M = Pa = 6232\,(96) = 598{,}300$ LB-IN

$V = P = 6323$ LB

$f_b = M/S = \dfrac{598.300}{306.3} = 1953$ PSI

$f_b = 1953$ PSI < 2253 PSI ✓

$f_v = 1.5\,V/A = \dfrac{1.5\,(6232)}{111.4} = 84$ PSI < 95 ✓

$\Delta_{D+L} = \dfrac{Pa}{24EI}\,(3\ell^2 - 4a^2)$

** $\Delta_{D+L} = \dfrac{6150\,(93.5)\left[3(283)^2 - 4\,(93.5)^2\right]}{24\,(1.8 \times 10^6)(2527)}$

$\Delta_{D+L} = 1.08''$

$\dfrac{\ell}{\Delta_{D+L}} = \dfrac{283}{1.08} = 262 > 240$ ✓

$\dfrac{\ell}{\Delta_L} = 262\left(\dfrac{779}{600}\right) = 340 \approx 360$ OK ✓

USE A 6¾" × 16½" 24 F GLULAM

CHECK END BEARING TRY A 5" LONG BEAM SEAT
R = 1.5 P = 6232 (1.5) = 9348 LB
$f_{c\perp} = \dfrac{9348}{6.75\,(5)} = 277$ PSI < 450 ✓

$* F_b' = C_d\,(0.968)(F_b) = 0.97\,(0.968)(2400) = 2253$ SI

** DIMENSIONS AND LOADS ARE LESS 5" END BE[illegible]NG

GLULAM ROOF SYSTEM DESIGN CALCULATIONS

LOADS

ROOF LOAD

DEAD

ROOFING, 3 PLY FELT & GRAVEL	5.5 PSF	
1/2" PLYWOOD SHEATHING	1.5 PSF	
2X4'S @ 16" O.C.	1.0 PSF	
	8.0 PSF	← TO STIFFENERS
4" X 10" @ 8' O.C.	1.0 PSF	
	9.0 PSF	← TO BEAM
3 1/8" X 18" @ 16' O.C.	1.0 PSF	
	10.0 PSF	← TO GLB

LIVE

T.A.	W_L
0-200	20 PSF
201-600	16 PSF
> 600	12 PSF

PLYWOOD T.A. = 32 FT^2

W_D = 7.0 PSF

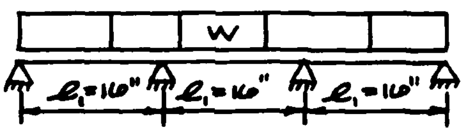
W_L = 20.0 PSF

27.0 PSF

TRY APA CD W/ EXT GLUE PLYWOOD
SPAN INDEX 32/16
FACE GRAIN || TO SUPPORTS

SPECIES GROUP 1; GRADE STRESS LEVEL S-2

A = 1.159 IN^2/FT	F_b = 1650 PSI
I = 0.006 IN^4/FT	F_v = 250 PSI
KS = 0.061 IN^3/FT	F_s = 75 PSI
Ib/Q = 2.746 IN^2/FT	E = 1.8×10^6 PSI



3-SPAN CONDITION

$\ell_2 = \ell_1 - 1.5" = 14.5"$

$$W_b = \frac{120\,F_b\,KS}{\ell_1^2} = \frac{120(1650)(0.061)}{(16)^2} = 47 \text{ PSF} > 27 \quad \text{O.K.} \checkmark$$

$$W_s = \frac{20\,F_s(Ib/Q)}{\ell_2} = \frac{20(75)(2.746)}{14.5"} = 284 \text{ PSF} > 27 \quad \text{O.K.} \checkmark$$

CHECK BENDING + SHEAR DEFL. $\ell_3 = 14.5 + 0.25 = 14.25$

BENDING → * $\Delta_b = \dfrac{W\ell_3^4}{1743\ EI} = \dfrac{1(14.75)^4}{1743(1.8\times10^6)(0.006)} = 0.00251''$

SHEAR → * $\Delta_s = \dfrac{W\,Ct^2\ell_3^2}{1270\ EI} = \dfrac{1(60)(0.5)^2(14.5)^2}{1270(1.8\times10^6)(0.006)} = 0.00023''$

TOTAL → * $\Delta_{b+s} = \Delta_b + \Delta_s = 0.00251 + 0.00023 = 0.00274''$

$\Delta_{ALLOW} = \dfrac{\ell}{240} = \dfrac{16}{240} = 0.0667''$

$W = \dfrac{\Delta_{ALLOW}}{\Delta_{b+s}} = \dfrac{0.0667''}{0.00274''} = 24.3$ PSF

$W = \dfrac{W_D}{2} + W_L = \dfrac{7.0}{2} + 20 = 23.5 < 24.3$ O.K. ✓

2"X4" STIFFENER   SPAN 8'-0"   T.A. = 10.6

DL = 8.0 PSF
LL = 20 PSF

$W_D = \dfrac{16}{12}(8) = 10.6$ PLF

$W_L = \dfrac{16}{12}(20) = 27$ PLF

37.2 PLF

2"X4"

$A = 5.25\ IN^2$ $F_b = 1650$ PSI
$S = 3.06\ IN^3$ $F_v = 95$ PSI
$I = 5.36\ IN^4$ $E = 1.7\times10^6$

$M = \dfrac{W\ell^2}{8} = \dfrac{37.2(8')(96'')}{8}$

M = 3,571 LB-IN

$V = \dfrac{W\ell}{2} = \dfrac{(37.2(8')}{2} = 149$ LB

$f_b = \dfrac{M}{S} = \dfrac{3571}{3.06} = 1167$ PSI

$f_b = 1167$ PSI < 1650 ✓

$f_v = \dfrac{1.5V}{A} = \dfrac{1.5(149)}{5.25} = 43$ PSI

$f_v = 43$ PSI < 95 PSI ✓

USE 2"X4" D.F. No.2 OR BETTER @ 16" O.C.

$\Delta_{D+L} = \dfrac{5W\ell^4}{384\ EI} = \dfrac{5(37.2)(8)(96)^3}{384(1.8\times10^6)(5.36)}$

$\Delta_{D+L} = 0.355''$

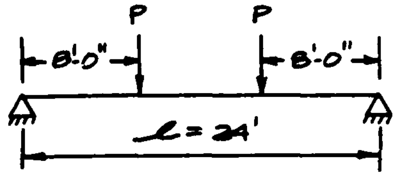
$\dfrac{\ell}{\Delta_{D+L}} = \dfrac{96}{0.355} = 270 > 240$ ✓

$\dfrac{\ell}{\Delta_{D+L}} = 270\left(\dfrac{37.2}{27}\right) = 372 > 360$ ✓

* DEFLECTIONS BASED ON A UNIT LOAD (i.e. W = 1 PSF)

GLUE LAMINATED BEAM   SPAN 24'-0", T.A. = 384 FT^2

$DL = 10$ PSF
$LL = 16$ PSF
$W_D = 16'(10) = 160$ PLF
$W_L = 16'(16) = \underline{256}$ PLF
416 PLF



$a = 8'$
$\ell = 24'$
$P = W(8') = 416(8) = 3328$ LB

TRY A 3 1/8" X 18" 24 F GLULAM

$A = 56.3$ IN^2 * $F_b' = 2304$ PSI
$S = 168.8$ IN^3 $F_v = 165$ PSI
$I = 1519$ IN^4 $E = 1.8 \times 10^6$ PSI
$C_d = 0.96$ $F_{c\perp} = 450$ PSI

DURATION OF LOAD ADJ = 1.25
REDUCTION FOR 2 POINT LOADING = 0.968

$$M = P(a) = 3328(96") = 319{,}500 \text{ LB}$$

$$V = P = 3328 \text{ LB}$$

$$f_b = \frac{M}{S} = \frac{319500}{168.8} = 1{,}892 \text{ PSI}$$

$$F_b = 2304(0.968)(1.25) = 2788' \text{ PSI} > 1892 \checkmark$$

$$f_v = \frac{1.5\ V}{A} = \frac{1.5(3328)}{56.3} = 89 \text{ PSI} < 165 \checkmark$$

$$\Delta_{D+L} = \frac{Pa}{24EI}(3\ell^2 - 4a^2)$$

$$\Delta_{D+L} = \frac{3328(96)[3(288)^2 - 4(96)^2]}{24(1.8 \times 10^6)(1519)}$$

$$\Delta_{D+L} = 1.03"$$

$$\frac{\ell}{\Delta_{D+L}} = \frac{288}{1.03} = 279 > 240 \checkmark$$

$$\frac{\ell}{\Delta_L} = 279\left(\frac{416}{256}\right) = 453 > 360 \checkmark$$

<u>USE A 3 1/8" X 18" 24 F GLULAM</u>

CHECK REACTION $R = 1.5P = 1.5(3328) = 4992$ LB

USE A 5" LONG BEAM SEAT $f_{c\perp} = \frac{4992}{5(3.125)} = 319$ PSI

$f_{c\perp} = 319$ PSI < 450 ✓

*$F_b' = C_d F_b = 0.96(2400) = 2304$ PSI

SAWN BEAM SPAN 16'-0" T.A. = 128 FT^2

DL = 9 PSF
LL = 20 PSF
W_D = 8'(9) = 72 PLF
W_L = 8'(20) = 160 PLF
232 PLF

$M = \frac{W\ell^2}{8} = \frac{232(16)(192")}{8} = 89100$ LB-IN

M = 89100 LB-IN

$V = \frac{W\ell}{2} = \frac{232(16)}{2} = 1856$ LB

TRY A 4"x10" D.F. No. 1

A = 32.4 IN^2 F_b = 1500 PSI
S = 49.9 IN^3 F_v = 95 PSI
I = 230.8 IN^4 E = 1.8×10^6 PSI

$f_b = \frac{M}{S} = \frac{89100}{49.9} = 1785$ PSI

f_b = 1785 < 1500 x 1.25 = 1875 PSI ✓

DURATION OF LOAD ADJUSTMENT ALLOWED FOR ROOFS

$f_v = \frac{1.5V}{A} = \frac{1.5(1856)}{32.4} = 86$

f_v = 86 PSI < 95 (1.25) = 119 PSI ✓

FOR DEFL. CALC'S USE SPAN LESS GLB WIDTH + BEARING WIDTH → $\ell = 16' - \frac{3.1" + 2"}{12} = 15.75'$

$\Delta_{D+L} = \frac{5W\ell^4}{384\,EI} = \frac{5(232)(15.8)(187)^3}{384(1.8 \times 10^6)(230.8)}$

$\Delta_{D+L} = 0.739"$

USE 4"x 10" D.F. No.1

$\frac{\ell}{\Delta_{D+L}} = \frac{187}{0.739} = 252 < 240$ OK ✓

$\frac{\ell}{\Delta_L} = 252\left(\frac{232}{160}\right) = 366 < 360$ ✓

END REACTION V = 1856 LB

USE A KC METALS H410 HEAVY JOIST HANGER

V_{ALLOW} = 1875 LB > 1856 LB ✓

USE 14 - N16 NAILS TO GLB
6 - 10d NAILS TO 4 X 10

REFERENCES

C-1. **Uniform Building Code**, International Conference of Building Officials, Whittier, CA, 1979.

C-2. American Institute of Timber Construction, **Timber Construction Manual,** Second Edition, John Wiley and Sons, Inc., New York, 1974.

C-3. Western Wood Products Association, **Product Use Manual,** revised November 1978.

C-4. **National Design Specification for Wood Construction**, 1977 Edition, National Forest Products Association.

C-5. **Glulam Systems**, American Institute of Timber Construction, Englewood, CO, 1978.

C-6. **Plywood Design Specification**, revised April 1978, American Plywood Association.

C-7. **Handbook of Structural Designs & Load Values**, KC Metals, Inc., San Jose, CA, 1981.

APPENDIX D

Glulam Analysis

Appendix D
GLULAM ANALYSIS

INTRODUCTION

The test program reported in Section 3 consisted of seven load tests to failure of two sizes of glulam beams. The purpose of these tests was to develop data on these particular beams, in both shored and unshored configurations, with respect to the failure loads, modes of failure, and the performance of the end bearing connections. In order to satisfy the overall goal of the program to develop a prediction methodology for the purpose of evaluating and upgrading of existing structures, these data must be extended to include as many of these types of beams as possible. Unfortunately, because of the characteristics associated with timber, it is not possible to reasonably predict performance with a small sample base. It is necessary to address such areas as statistics of material variability, moisture content, duration and type of loading, size and depth effects, and the probability aspects in order to predict behavior. Accordingly, additional data on tests conducted by others on these types of beams were obtained and analyzed, and combined with the data from the three unshored (base case) tests conducted under the current program (Section 3).

In 1969 and 1970 the U.S. Department of Agriculture Forest Products Laboratory conducted 41 flexural tests on large glued laminated beams and published the results in research papers FPL 113 (Ref. D-1) and FPL 146 (Ref. D-2). These 41 tests plus the three tests conducted by Scientific Service, Inc. are shown in Table D-1. Included in this table are the pertinent dimensions, specifications used for manufacture, timber species, and allowable bending stresses.

With respect to the Forest Products Laboratory tests, the selection of the tension laminations in the zone of maximum bending moment was at or near the

TABLE D-1: TEST SPECIMEN DATA

Beam Number	Size (in.)	Length (ft)	Wood Species	AITC Tension Laminate Spec.	Report Number	MFG Spec	Listed Bending Stress Rating (psi)	Recommended Allowable Bending Stress (psi)
1 - 5	$5\frac{1}{8}$ x $24\frac{3}{4}$	40	Douglas Fir, Coast	301-67	FPL-113	(1)	2600	<2600
6 - 10	$5\frac{1}{8}$ x $24\frac{3}{4}$	40	Douglas Fir, Coast	301+	FPL-113	(1)	2600	2600
11 - 15	$5\frac{1}{8}$ x $24\frac{3}{4}$	40	Southern Pine	301-67	FPL-113	(3)	2600	<2600
16 - 20	$5\frac{1}{8}$ x $24\frac{3}{4}$	40	Southern Pine	301+	FPL-113	(3)	2600	2600
21 - 23	9 x $31\frac{1}{2}$	50	Douglas Fir, Coast	301-67	FPL-113	(1)	2600	<2600
24 - 26	9 x $31\frac{1}{2}$	50	Southern Pine	301-67	FPL-113	(3)	2600	<2600
36 - 40	$5\frac{1}{8}$ x $24\frac{3}{4}$	40	Southern Pine	301A-69	FPL-146	(3)	2600	2600
41 - 45	$5\frac{1}{8}$ x $24\frac{3}{4}$	40	Douglas Fir, Coast	301A-69	FPL-146	(1)	2600	2400
46 - 50	$5\frac{1}{8}$ x $24\frac{3}{4}$	40	Douglas Fir, Int. North	301A-69	FPL-146	(2)	2600	2400
A, B	$6\frac{3}{4}$ x $16\frac{1}{2}$	24	Douglas Fir	302-24	SSI	(4)	2400	2400
C	$3\frac{1}{8}$ x 18	24	Douglas Fir	302-24	SSI	(4)	2400	2400

(1) Combination A, "Standard Specification for Structural Glue Laminated Douglas Fir (Coast Region) Timber"

(2) Combination 26F, "Standard for Structural Glued Laminated Members Assembled with WWPA Grades of Douglas Fir and Larch Lumber"

(3) Combination A-2, "Standard Specifications for Structural Glued Laminated Southern Pine Timber"

(4) Combination 24F-V3, AITC 117-79, "Standard Specifications for Structural Glued Laminated Timber of Softwood Species"

minimum quality permitted by the lamination specifications 301-67, 301+, and 301A-69 (see Table D-1). These specifications did not differ significantly one from another.

An attempt was also made to use the worst face of the boards as the bottom face of the beam. Finger joints in the tension laminations of the beams were intentionally placed at least 1.4 feet outside the area of maximum positive bending moment, for the FPL 113 test beams (beam Nos. 1 through 26, Table D-1), and a minimum of 2.0 feet outside the area of maximum positive bending moment for the FPL 146 test beams, (beam Nos. 36 through 50, Table D-1).

It must be kept in mind that, although an attempt was made to introduce maximum flaws, these were "test specimens" and may well not have the flaws common in normal manufacturing.

ANALYSIS OF TEST DATA

The allowable design stress assumes a simply supported beam 12 inches deep with a span-to-depth ratio of 21 to 1 and carrying a uniformly distributed load. In addition, the allowable design stress assumes a normal, 10-year, duration of load. The analysis of data, which follows, will present the data in two ways: first, the actual test data corrected to a common 12% moisture content and a best fit line (least squares regression line) drawn; then, the best fit line will be corrected to account for the differences between the test specimens and the assumptions made for the allowable design stresses that have been mentioned above.

Prior to further analysis of the data, a brief description of three factors that directly influence the strength of wood appears to be in order. These three factors are duration of loading, the size or depth adjustment, and the uniform load vs third point loading adjustment. Loading rates can greatly affect the strength of wood, by as much as 200%. As stated in Ref. D-3, "Based on the existing evidence of variation of strength with duration of loading, strength properties determined in tests that last usually from 6 to 8 minutes can be converted to other durations of

loading by traditional methods. Strength properties for the so-called normal loading conditions* may be determined by multiplying standard strength properties by the factor 1/1.6; the same can be done for other loading conditions using corresponding factors." This allowable load vs duration relationship is presented in Figure D-1.

The second factor affecting the strength of a glulam is the size or depth effect factor. This factor will be treated here as being made up of a combination of two terms, a depth correction and a span-to-depth correction. Allowable stresses used in glulam design for flexure or bending are given in terms of a 12-inch deep beam, for other beam depths the allowable bending stress must be modified. The reason for this is that as the depth of a member increases, so does the probability that a flaw or region of low strength will occur. Gurfinkel (Ref. D-4) and Hoyle (Ref. D-5) discuss this correction for strength for depths greater than 12 inches. The formula that follows describes this reduction in strength as C_d.

$$C_d = \left(\frac{12}{d}\right)^{1/9}$$

where C_d is the strength reduction factor.

This strength reduction factor is also corrected to account for beam length by use of the span-to-depth ratio. The span-to-depth ratio for the 24½-inch deep, 40-foot long and the 31½-inch deep, 50-foot long beams is 19.8 and 19.0, respectively, an increase of 1.008 times C_d. The span-to-depth ratio for the 16½-inch deep, 24-foot long beam was 17.4, and for the 18-inch deep, 24-foot long beam was 16.0. (See Ref. D-5 for further discussion of this reduction.)

The third and final term to be accounted for is a loading condition adjustment reduction: in testing these specimens two loads were applied at approximately the third points, and a loading condition adjustment reduction of 3.2% is recommended to account for the high region of stress, which is larger than that for a simply supported beam subjected to a uniform load.

* Normal loading is a 10-year continuous or cumulative load over the life of the structure.

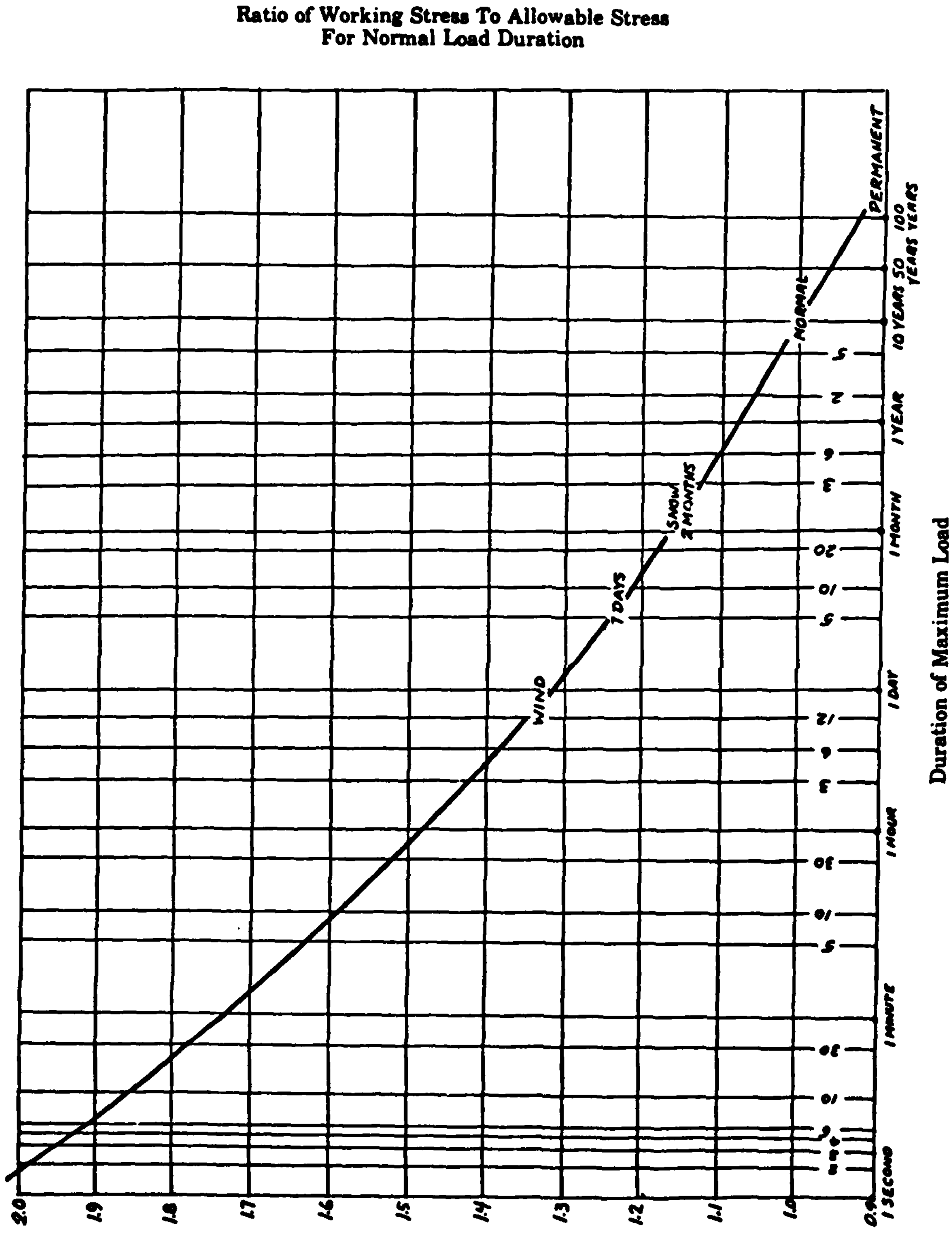


Fig. D-1. Allowable Load vs Duration of Load Relationship (from Ref. D-5).

For the 24½-inch glulams tested, the total correction is:

$$1/(1.6)0.92(1.008)0.97 = 1/1.44$$

For the 31½-inch glulams, the total correction is:

$$1/(1.6)0.90(1.008)0.97 = 1/1.40$$

For the 16½-inch glulams, the total correction is:

$$1/(1.51)0.97(1.015)0.97 = 1/1.44$$

For the 18-inch glulams, the total correction is:

$$1/(1.45)0.96(1.015)0.97 = 1/1.40$$

Probability Analysis Results of the FPL 146 Test Data (Beams 36 through 50)

Forest Products data from these tests are presented on Table D-2 in terms of modulus of the rupture at failure. For comparison, the modulus of rupture for all the beams were adjusted to 12% moisture content and a common 24½-inch member depth. The adjusted results for the fifteen glulams tested in this group were plotted on normal probability distribution, and then on log normal probability distribution paper. A review of these plots showed that the log normal probability distribution better described the failure strength of these glulams, and therefore, it was used throughout the remainder of this analysis.

On Figures D-2 through D-5 the normal probability distribution of the modulus of rupture for each of the three groups of glulams tested has been plotted (upper set of curves). The actual test data adjusted only by a common 12% moisture content have been plotted, and the best fit line or least squares regression line (upper solid line) drawn. The upper and lower 50% confidence limits (dashed lines) have been added above and below the solid best fit line. Since only a few tests were conducted on each type of glulam and there can be a great deal of uncertainty

TABLE D-2

MODULUS OF RUPTURE AT FAILURE
FOR THE FPL-146 BEAM TESTS

Beam Number	Modulus of Rupture* (psi)	Cause of Failure
Southern Pine		
36	6700	End joint
37	7600	Wood
39	7820	Wood
40	8220	Wood
38	8660	Wood
Douglas Fir (Coast)		
42	4520	Wood and end joint
44	5360	Wood
43	5380	Wood
45	5920	Wood
41	6360	End joint
Douglas Fir (Interior North Region)		
47	3520	Pre-existing compressive failure
49	4880	Wood
48	5320	End joint
50	5550	End joint
46	6560	Wood

* Adjusted to 12% moisture content.

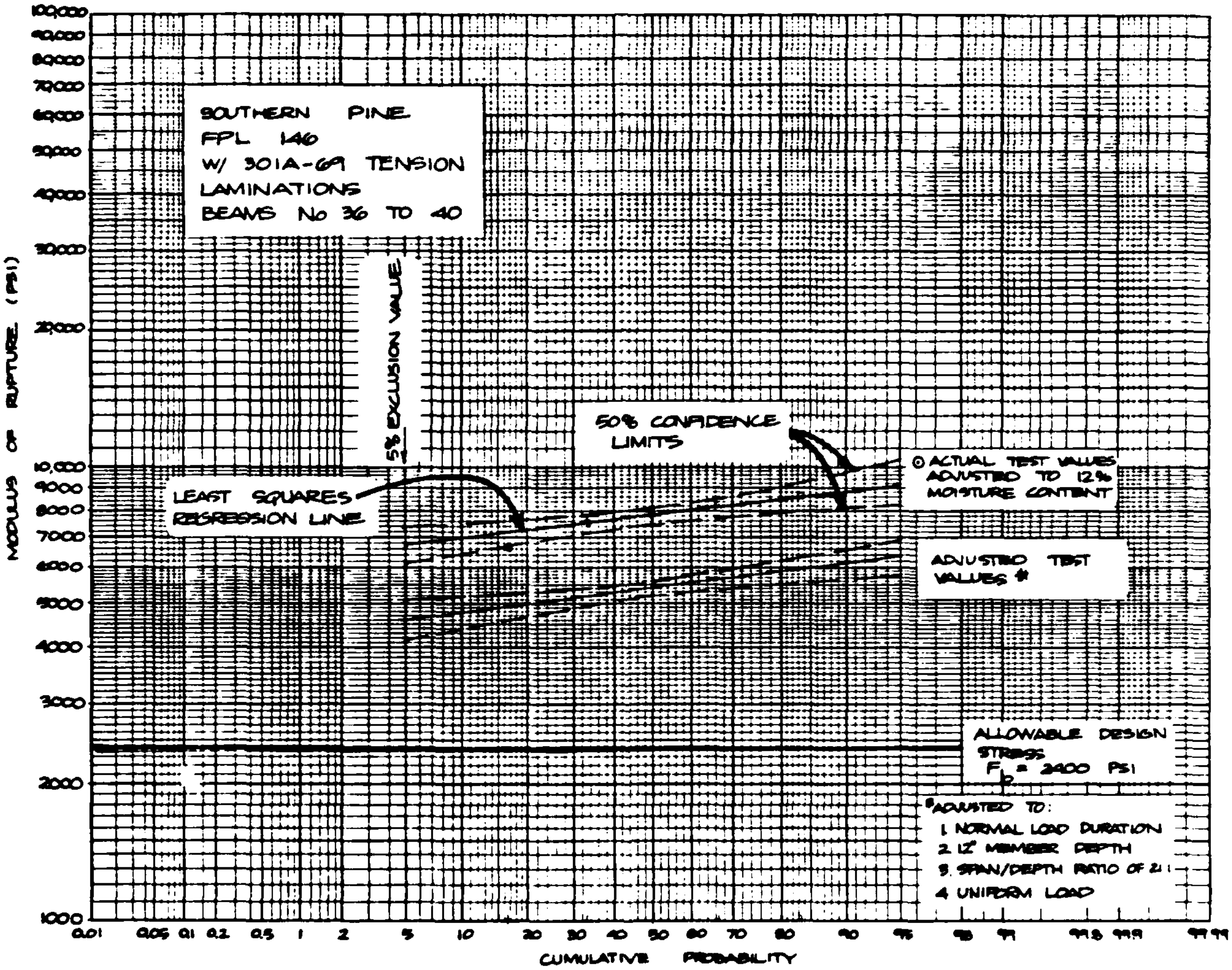


Fig. D-2. Modulus of Rupture vs Cumulative Probability for Five Southern Pine Glulam Beams With 301A-69 Tension Laminations.

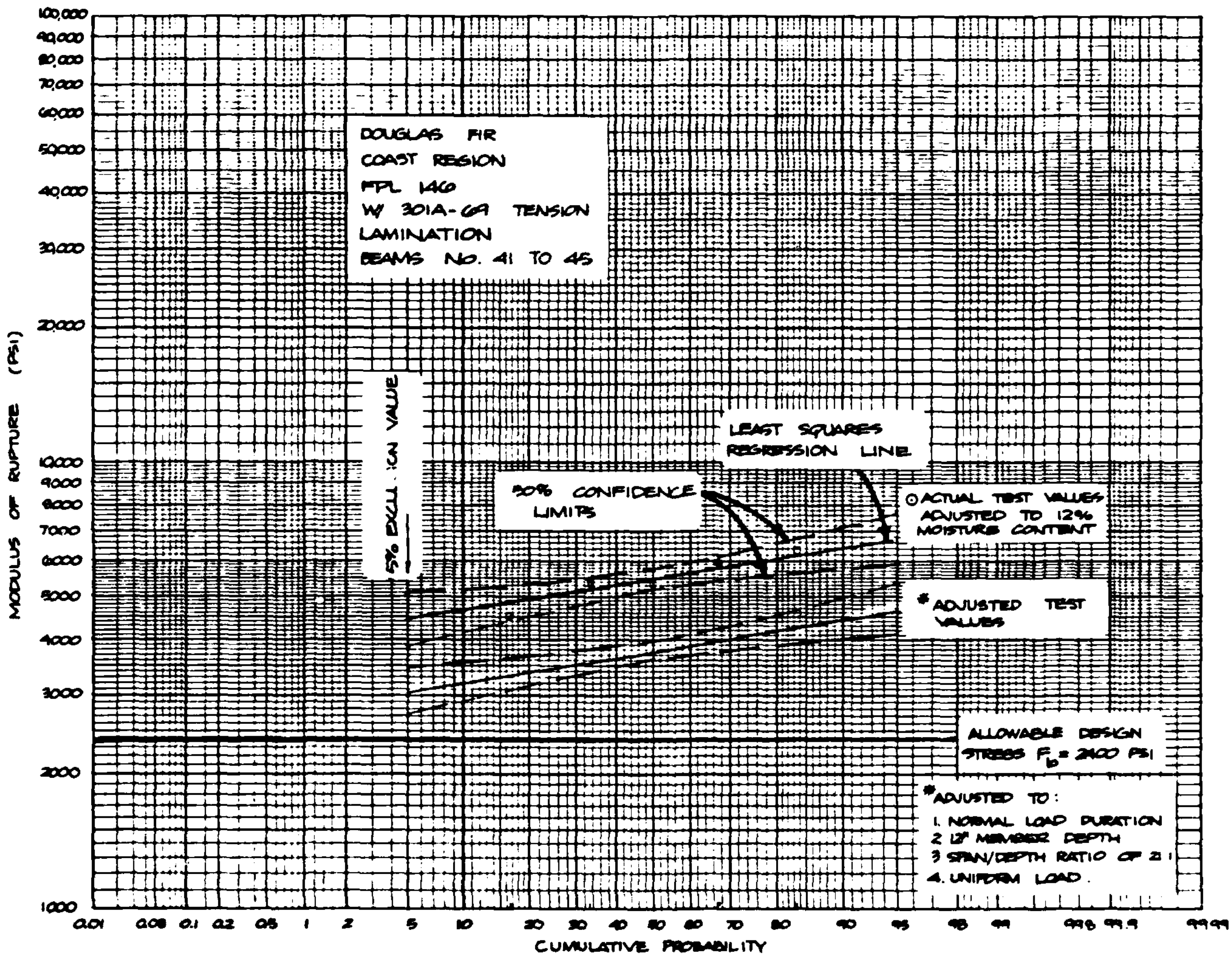


Fig. D-3. Modulus of Rupture vs Cumulative Probability for Five Douglas Fir (Coast Region) Glulam Beams With 301A-69 Tension Laminations.

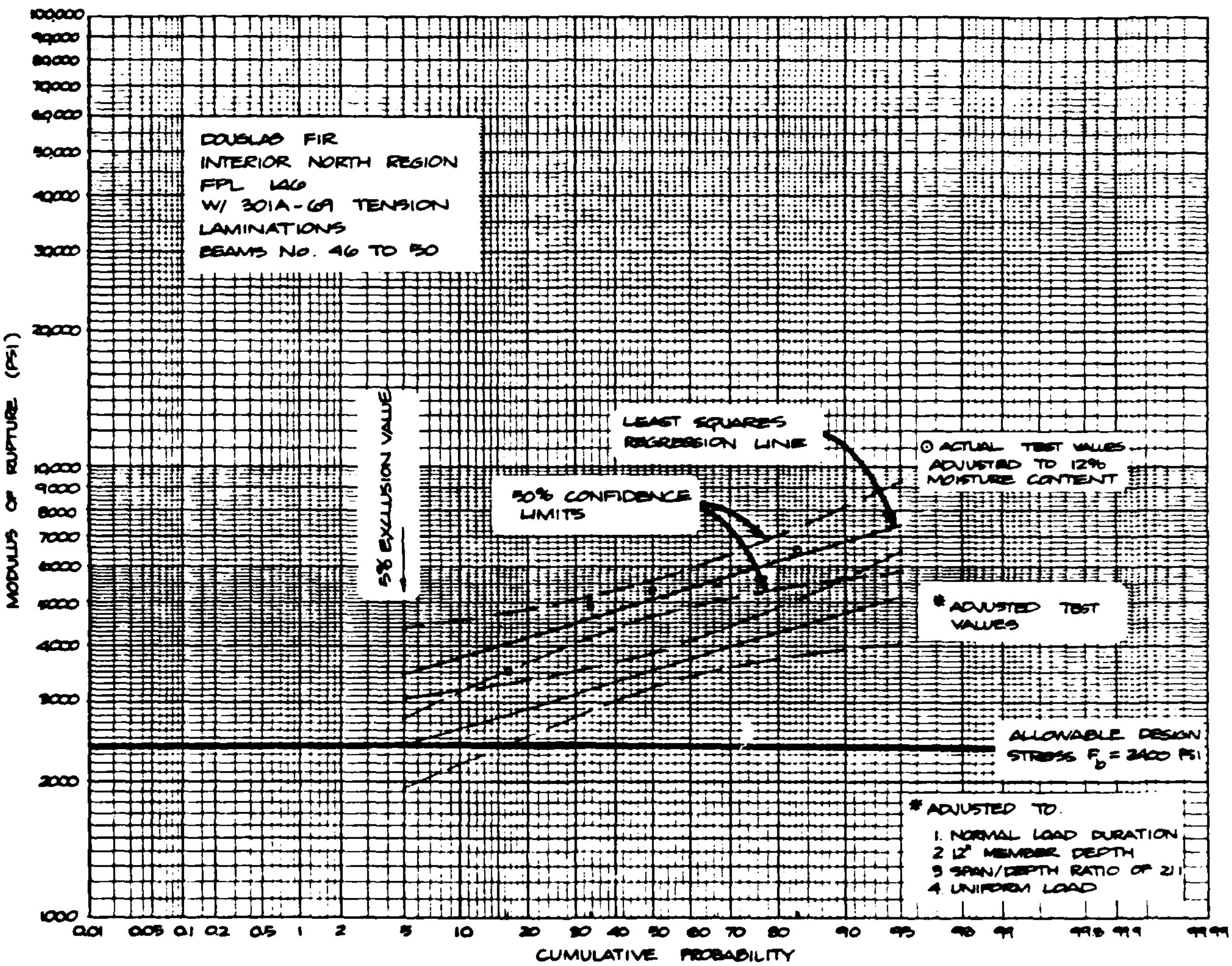
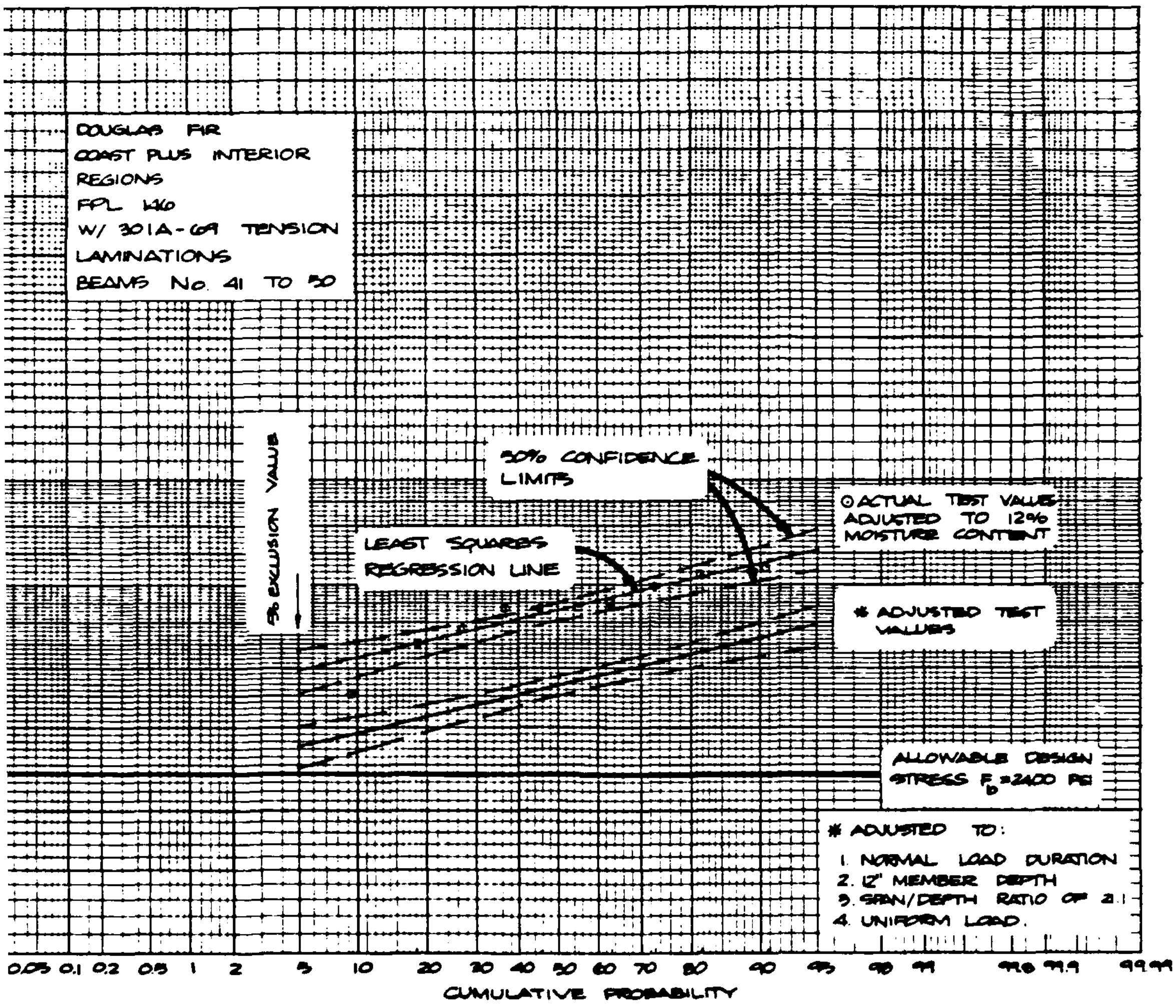


Fig. D-4. Modulus of Rupture vs Cumulative Probability for Five Douglas Fir (Interior North Region) Glulam Beams With 301A-69 Tension Laminations.



D-5. Modulus of Rupture vs Cumulative Probability for Ten Douglas Fir (Combined Coast Plus Interior North Region) Glulam Beams With 301A-69 Tension Laminations.

whether the regression line represents the true probability distribution, the 50% confidence limits have been added to account for the uncertainty in the regression line. In effect, the true regression line has a 50% probability of being within the upper and lower 50% confidence limits. The least squares regression line and confidence limits, reduced by a common factor of 1/1.44, were again plotted on Figures D-2 through D-5 (lower set of curves), for the purpose of evaluation with the stated allowables. Based upon the conclusions of research report FPL 146, the allowable design flexural stress for these glulams should be 2,400 psi. The allowable design load of 2,400 psi is shown on Figures D-2 through D-5 as the lower horizontal darkened line. The factor of safety is the ratio of either the lower 50% confidence limit and the allowable design stress or the lower regression line and the allowable design stress. The wood industry has traditionally used the 5% exclusion value to calculate the factor of safety. The factor of safety against faiure has been presented as two values: (1) the ratio of 5% exclusion value on the regression line to the allowable design stress, or (2) the ratio of the 5% exclusion value on the lower 50% confidence limit to the allowable design stress.

Conclusions Based Upon FPL 146 Test Results

Based upon the probabilistic approach presented in this report, it can be concluded that the Southern Pine beams (Nos. 36 to 40) are adequate for their intended design allowable of 2,600 psi fiber stress. The Southern Pine glulams have a factor of safety against failure of 1.74 with 50% confidence that only 5% of the members fabricated in this manner will have a factor of safety less than 1.74 (this is referred to as the 5% exclusion level). The confidence limit corrects for the uncertainty in the predicted 5% exclusion value because of the limited number of tests conducted.

Ten Douglas Fir glulam beams (Nos. 41 to 50) were tested, of which five were fabricated from Coast Region Douglas Fir and the remainder from Interior Region Douglas Fir. These members were initially designed as having a 2,600 psi fiber stress; as a result of these tests, however, it was recommended in the conclusion of the FPL report that the allowable fiber stress be reduced to 2,400 psi. Using the fiber stress recommended by FPL, and the probabilistic methods mentioned above, the factor of safety against failure for these glulams is 1.13 and 0.80 for the Coast

Region and Interior Region Douglas Fir glulams, respectively, see Table D-3. These factors of safety were based on a predicted 5% exclusion value and a 50% confidence limit. In reviewing the fabrication details of the glulams tested, a high degree of similarity was noticed between the Douglas Fir Coast and the Douglas Fir Interior glulam beams. The test results for both these groups of glulams were combined, resulting in the probability distribution shown in Figure D-5. By inspection, there appear to be no statistical differences between the two groups of glulams. The test data show a good fit to the least squares regression line.

Probability Analysis Results for the FPL 113 Test Data

Forest Products data from these tests are presented on Table D-4 in terms of the modulus of rupture at failure. For comparison the modulus of rupture for all the beams has been adjusted to a 12% moisture content and a 24-3/8 inch member depth. These results were then plotted on log normal probability distribution paper and are shown on Figures D-6 through D-9 (upper set of curves). The best fit line (solid line) for the data was drawn. The upper and lower 50% confidence limits (dashed lines) have been added above and below the best fit line. The best fit line and its associated confidence limits have been reduced by the common factor of 1/1.44 to allow comparisons to be drawn with the allowable design stress (lower set of curves). These 26 glulams were designed and manufactured for an intended design allowable of 2,600 psi fiber stress. It should be noted that in the conclusions of FPL 113 it was recommended that the design stress for these beams be less than 2,600 psi, although how much lower than 2,600 psi was not given. For lack of a better number, the allowable design stress of 2,600 psi was used for comparison, and is shown on Figures D-6 through D-9 as the dark horizontal line. Two factors of safety against failure have been calculated, one for the best fit 5% exclusion value, and the other for the lower 50% confidence 5% exclusion value. The 5% exclusion values and their associated factors of safety are summarized in Table D-5.

Conclusions Based Upon FPL 113 Test Results

Based on probabilistic approach presented in this report, the 5% exclusion values for the modulus of rupture used to calculate the factors of safety are shown on Table D-5. Each of these values has been adjusted for comparisons to the allowable design stress. The factors of safety against failure for each type of

TABLE D-3

FACTOR OF SAFETY AND MODULUS OF RUPTURE DETERMINATION
FOR LARGE GLUED-LAMINATED BEAMS
TESTED IN FPL-146 REPORT

Glued-Laminated Beam Test Series Description	Modulus of Rupture (psi) 5% Exclusion		Factor of Safety, F.S. 5% Exclusion	
	Best Fit	Lower 50% Confidence Limit	Best Fit	Lower 50% Confidence Limit
1	2	3	4	5
1. Southern Pine FPL Beams 36 to 40	4601	4177	1.92	1.74
2. Douglas Fir (Coast) FPL Beams 41 to 45	3074	2705	1.28	1.13
3. Douglas Fir (Interior) FPL Beams 46 to 50	2406	1913	1.00	0.80
4. Douglas Fir (Coast Int.) FPL Beams 41 to 50	2715	2462	1.13	1.03
5. Douglas Fir SSI Base Case Tests	3317	2746	1.38	1.14

TABLE D-4

MODULUS OF RUPTURE AT FAILURE
FOR THE FPL-113 BEAM TESTS

Beam Number	Modulus of Rupture* (psi)	Cause of Failure
Douglas Fir (Coast)		
	301-69 Tension Laminations	
4	4140	Wood
5	4430	Wood
2	5000	End joint
22**	5210	Wood
3	5260	Wood
21**	5464	Wood
1	5650	End joint
23**	5834	End joint
	301+ Tension Laminations	
10	5220	End joint
7	5890	End joint
8	6150	Wood
6	6180	Wood
9	6300	End joint
Southern Pine		
	301-67 Tension Laminations	
14	3950	Wood
11	4800	End joint
25**	4836	Wood
26**	4867	End joint
15	4910	Wood
13	4920	Wood
24**	5247	Wood
12	6250	End joint
	301+ Tension Laminations	
16	4180	Wood
17	4280	End joint
19	4960	End joint
20	6390	End joint
18	7350	Wood

* Based on 12% moisture content

** Adjusted from 31½-inch to 24½-inch depth

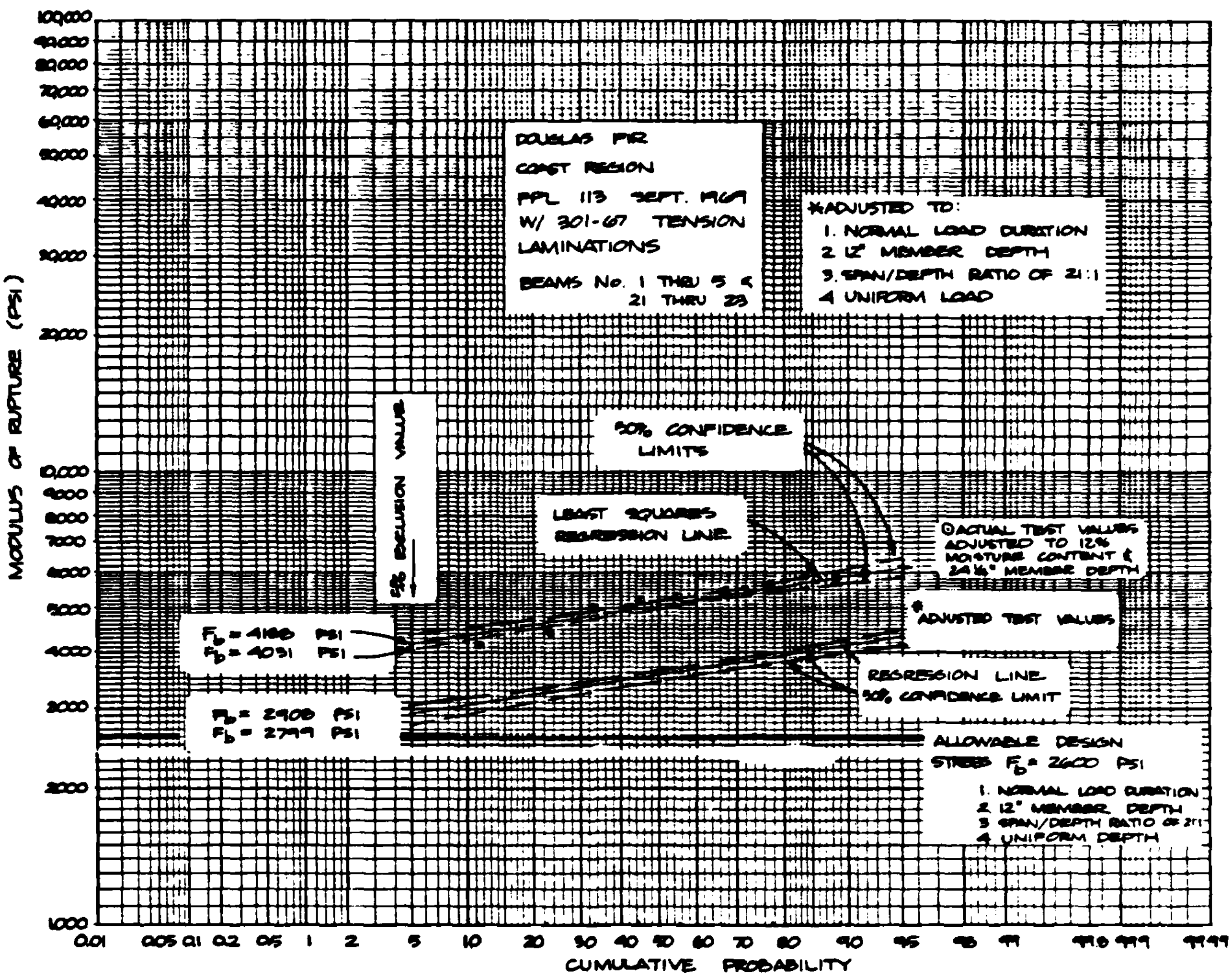


Fig. D-6. Modulus of Rupture vs Cumulative Probability for Five Douglas Fir (Coast Region) Large Glulam Beams With 301-67 Tension Laminations.

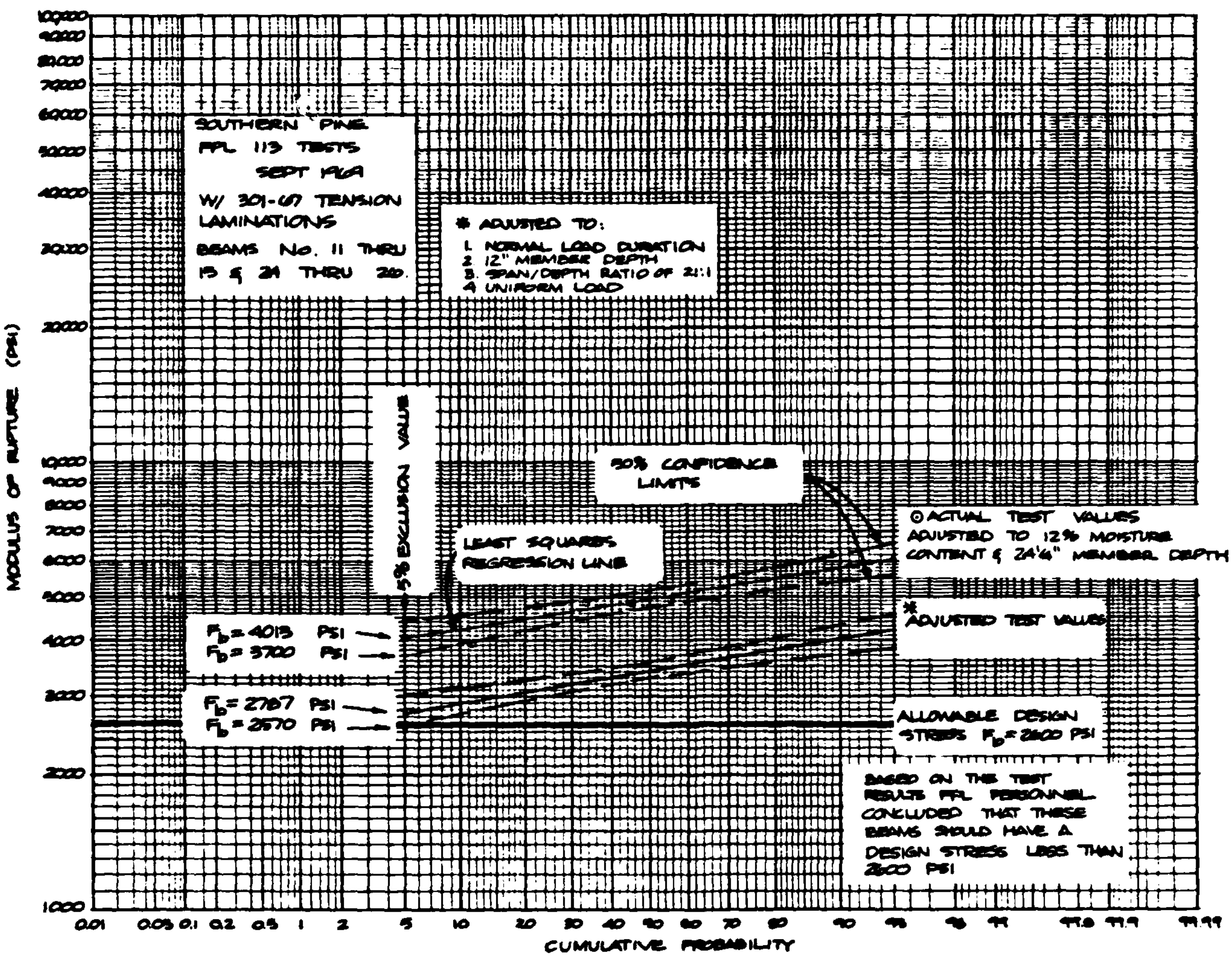


Fig. D-7. Modulus of Rupture vs Cumulative Probability for Eight Southern Pine Large Glulam Beams With 301-67 Tension Laminations.

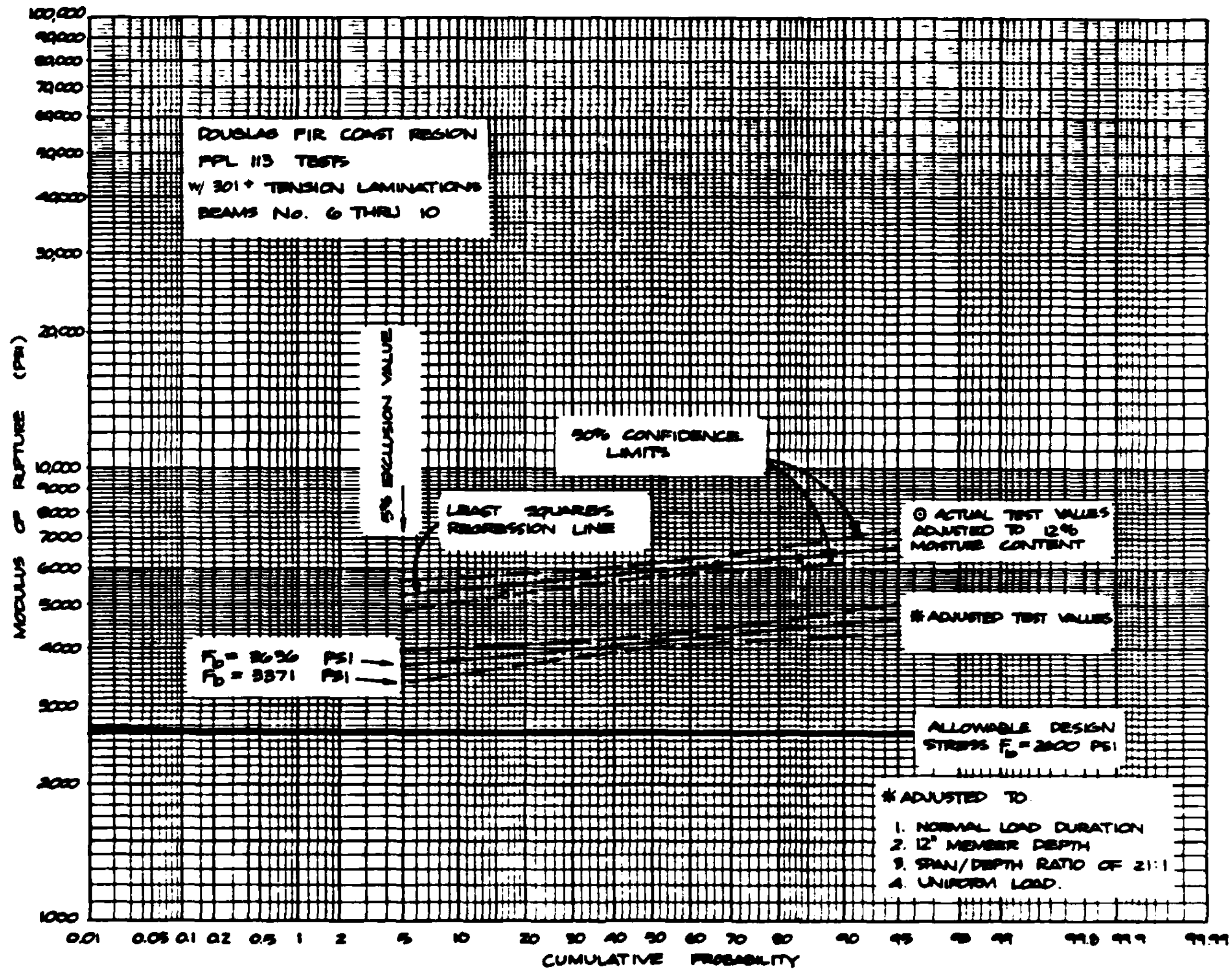


Fig. D-8. Modulus of Rupture vs Cumulative Probability for Five Douglas Fir (Coast Region) Large Glulam Beams With 301+ Tension Laminations.

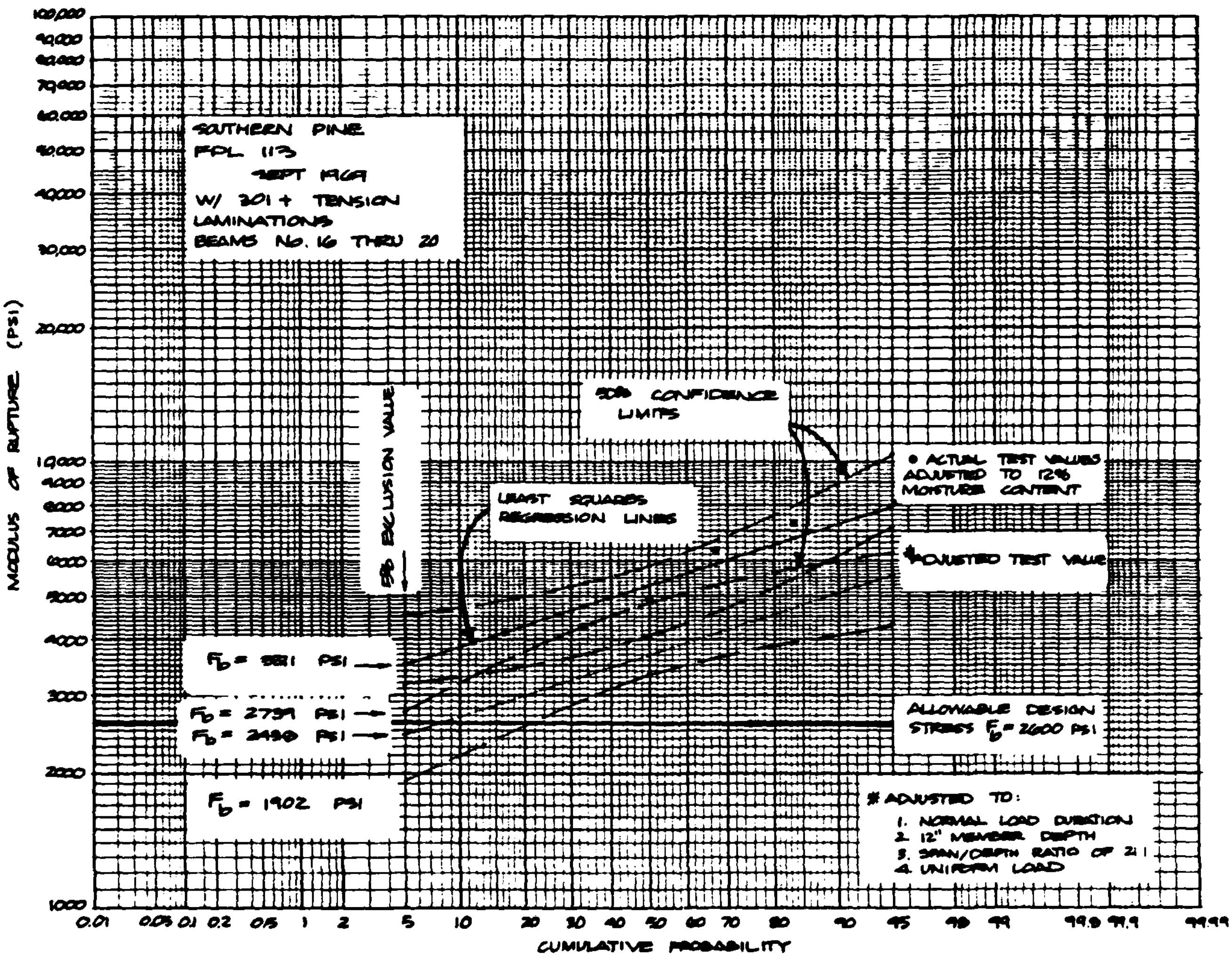


Fig. D-9. Modulus of Rupture vs Cumulative Probability for Five Southern Pine Large Glulam Beams With 301+ Tension Laminations.

TABLE D-5

FACTOR OF SAFETY AND MODULUS OF RUPTURE DETERMINATION
FOR LARGE GLUED-LAMINATED BEAMS
TESTED IN FPL-113 REPORT

Glued-Laminated Beam Test Series Description	Modulus of Rupture (psi) 5% Exclusion		Factor of Safety, F.S. 5% Exclusion	
	Best Fit	Lower 50% Confidence Limit	Best Fit	Lower 50% Confidence Limit
1	2	3	4	5
Douglas Fir, Coast Region Beams 1 to 5, 21 to 23	2908	2799	1.12	1.07
Douglas Fir, Coast Region Beams 6 to 10	3636	3371	1.51	1.29
Southern Pine Beams 11 to 15, 24 to 26	2787	2570	1.07	0.98
Southern Pine Beams 16 to 20	2438	1902	0.93	0.73

The factor of safety against failure was computed as:

$$F.S. = \frac{MOR}{2600\ \text{psi}}$$

where 2600 psi was the allowable design flexural stress.

glulam tested have also been computed and these are shown in columns 4 and 5 of Table D-5. The factor of safety against failure indicated is the ratio of the modulus of rupture to the allowable bending stress of 2,600 psi.

SSI Base Case Test Results

Three simply supported (unshored) glulams were tested to failure and are reported in Section 3 of this report (beams A, B, and C, Table D-1). The resulting probability distribution for these three tests is shown in Figure D-10. The best fit line and 50% confidence limits are shown in this figure. These three beams all had an allowable (design) flexure stress of F_b = 2,400 psi; the resulting factor of safety against failure based on these tests is 1.14 (see bottom of Table D-3) based on the lower 50% confidence limit and a 5% exclusion value, and 1.38 based on the best fit and a 5% exclusion value.

CONCLUSION

A summary of the modulus of rupture and the factor of safety for each of the glulam beam groups analyzed is shown in Table D-6. For this tabulation, the best fit curves with the 5% exclusion values were used. These values are used, rather than the 50% confidence values, since they are more appropriate for use in predicting the performance of existing structures for shelter upgrading.

A review of Table D-6 indicates a considerable spread in the factor of safety, 0.93 to 1.92. A factor of safety of 1.3 is intended by the industry for timber members, and Ref. D-1 states that it was assumed for the FPL tests. Based on data included in this investigation, it is our conclusion that, after adjustments are made for moisture content, size and depth effects, and loading duration, a factor of safety of only 1.0 is applicable. The 1.0 factor, based on the 5% exclusion value, is conservative and consistent with the data and is appropriate for use in the prediction methodology for these types of members for loads excluding blast. As in previous work, a 1.6 factor of safety will be used for blast loading.

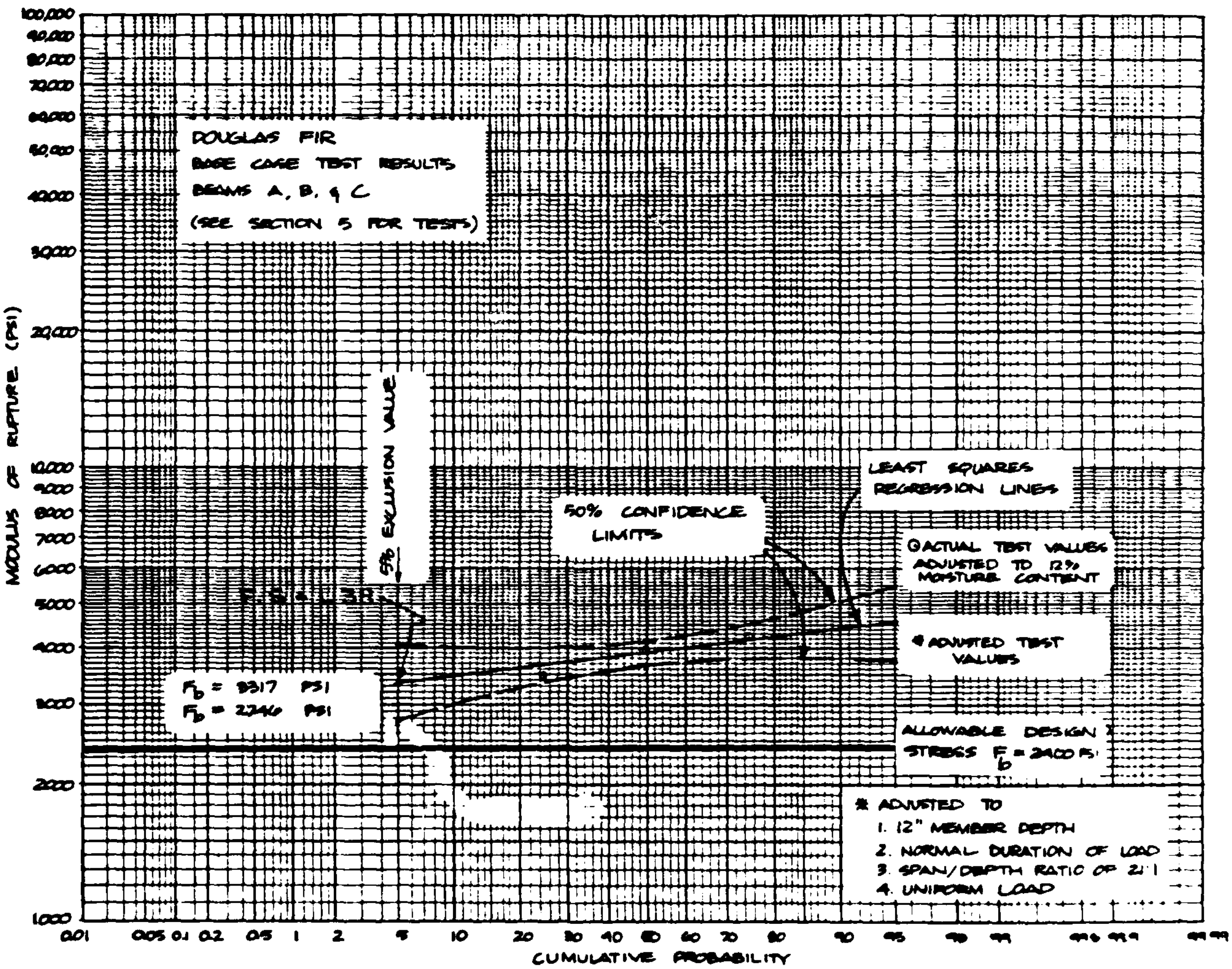


Fig. D-10. Modulus of Rupture vs Cumulative Probability for Three Douglas Fir, Large Glulam Beams.

TABLE D-6: SUMMARY OF MODULUS OF RUPTURE AND FACTORS OF SAFETY

Beam Number	Size (in.)	Length (ft)	Wood Species	Modulus of Rupture 5% Exclusion (psi)	Factor of Safety 5% Exclusion
1 - 5	5¼ x 24¼	40	Douglas Fir, Coast Region	2908	1.12
6 - 10	5¼ x 24¼	40	Douglas Fir, Coast Region	3636	1.51
11 - 15	5¼ x 24¼	40	Southern Pine	2787	1.07
16 - 20	5¼ x 24¼	40	Southern Pine	2438	0.93
21 - 23	9 x 31½	50	Douglas Fir, Coast Region	2908	1.12
24 - 26	9 x 31½	50	Southern Pine	2787	1.07
36 - 40	5¼ x 24¼	40	Southern Pine	4601	1.92
41 - 45	5¼ x 24¼	40	Douglas Fir, Coast Region	3074	1.28
46 - 50	5¼ x 24¼	40	Douglas Fir, Interior N. Region	2406	1.00
A, B	6¾ x 16½	24	Douglas Fir	3317	1.38
C	3⅛ x 18	24	Douglas Fir	3317	1.38

REFERENCES

D-1. **Large Glued-Laminated Timber Beams With Two Grades of Tension Laminations**, U.S.D.A Forest Service Research Paper FPL 113, Forest Products Laboratory, Madison, WI, September 1969.

D-2. **Large Glued-Laminated Beams With AITC 301A-69 Grade Tension Laminations**, U.S.D.A Forest Service Research Paper FPL 146, Forest Products Laboratory, Madison, WI, October 1970.

D-3. Gabrielsen, B.L., G. Cuzner, and R. Lindskog, **Blast Upgrading of Existing Structures**, SSI 7719-4, Scientific Service, Inc., Redwood City, CA, January 1979.

D-4. Gurfinkel, German, **Wood Engineering**, Southern Forest Products Association, New Orleans, LA, 1973.

D-5. Hoyle, Robert J., **Wood Technology in the Design of Structures**, Mountain Press Publishing Company, Missoula, MT, 1972.

DISTRIBUTION LIST

(One copy unless otherwise specified)

'ederal Emergency Management Agency
ıttn: Assistant Associate Director
for Research
[ational Preparedness Programs
Directorate
/ashington, D.C. 20472 (3)

)r. Michael A. Pachuta
ndustrial Protection Division
[ational Preparedness Programs
'ederal Emergency Management Agency
/ashington, D.C. 20472 (50)

[r. Phillip M. Smith
ıssociate Director
[atural Resources & Commercial
Services
)ffice of Science and Technology Policy
:xecutive Office Building
/ashington, D.C. 20500

)efense Technical Information Center
:ameron Station
.lexandria, VA 22314 (12)

[r. Carl Wiehle
)efense Intelligence Agency
.ttn: CKW DB-4C2
/ashington, D.C. 20301

irector, Defense Nuclear Agency
ttn: Technical Library
'ashington, D.C. 20305

irector, Defense Nuclear Agency
ttn: Mr. Tom Kennedy
'ashington, D.C. 20305

ssistant Secretary of the Army (R&D)
ttn: Assistant for Research
ashington, D.C. 20306

Director, Army Materials and Mechanics
Research Center
Attn: Technical Library
Watertown, MA 02172

Chief of Engineers
Department of the Army
Attn: ENGEME-RD
Washington, D.C. 20314

Director, U.S. Army Ballistic Research
Laboratory
Attn: Document Library
Aberdeen Proving Ground, MD 21005

Mr. William Taylor
Ballistic Research Laboratory
Aberdeen Proving Ground, MD 21005 (2)

Director, U.S. Army Engineer
Waterways Experiment Station
Attn: Document Library
P.O. Box 611
Vicksburg, MS 39180

Mr. W.L. Huff
USAE Waterways Experiment Station
P.O. Box 631
Vicksburg, MS 39180

Chief of Naval Research
Washington, D.C. 20306

Commanding Officer
U.S. Naval Civil Engineering Laboratory
Attn: Document Library
Port Hueneme, CA 93041

Civil Engineering Center AF/PRECET
Attn: Technical Library
Wright-Patterson Air Force Base
Dayton, OH 45433

Air Force Weapons Laboratory
Attn: SUL Technical Library
Kirtland Air Force Base
Albuquerque, NM 87117

Air Force Weapons Laboratory
Civil Engineering Division
Kirtland Air Force Base
Albuquerque, NM 87117

Mr. Lewis V. Spencer
National Bureau of Standards
Room C313 - Building 245
Washington, D.C. 20234

Mr. Samuel Kramer, Chief
Office of Federal Building Technology
Center for Building Technology
National Bureau of Standards
Washington, D.C. 20234

Dr. Rudolf J. Engelmann
Department of Energy
Department of Military Application
Washington, D.C. 20545

Dr. Barry Bowman
Lawrence Livermore National Laboratory
University of California
Box 808,
Livermore, CA 94550

Oak Ridge National Laboratory
Attn: Librarian
P.O. Box X
Oak Ridge, TN 37830

Emergency Technology Division
Oak Ridge National Laboratory
Attn: Librarian
Oak Ridge, TN 37830

Los Alamos Scientific Laboratory
Attn: Document Library
Los Alamos, NM 87544

Dr. Clarence R. Mehl
Division 1112
Sandia National Laboratories
Box 5800
Albuquerque, NM 87185

Mr. C. Wilton
Scientific Service, Inc.
517 East Bayshore
Redwood City, CA 94063 (2)

Mr. Richard Laurino
Center for Planning and Research
2483 E. Bayshore Road, Suite 104
Palo Alto, CA 94303

Mr. Fred Sauer
Physics International Company
2700 Merced Street
San Leandro, CA 94577

The Dikewood Corporation
1613 University Blvd, N.E.
Albuquerque, NM 87102

Applied Research and Associates
Attn: Cornelius J. Higgins
2601 Wyoming Blvd, Suite H-1
Albuquerque, NM 87112

Mr. Thomas E. Watermann
IITRI
10 West 35th Street
Chicago, IL 60616 (2)

Mr. Leo A. Schmidt
Institute for Defense Analyses
Program Analysis Division
400 Army-Navy Drive
Arlington, VA 22202

The RAND Corporation
Attn: Document Library
1700 Main Street
Santa Monica, CA 90401

Director,
Lovelace Foundation
5200 Gibson Blvd, S.E.
Albuquerque, NM 87108

Dr. William Chenault
Human Sciences Research, Inc.
Westgate Industrial Park
7710 Old Springhouse Road
McLean, VA 22102

Mr. Kenneth Kaplan,
30 White Plains Court
San Mateo, CA 94402

Mr. John Rempel
Center for Planning and Research
2483 E. Bayshore
Palo Alto, CA 94303

H.L. Murphy Associates
Box 1727
San Mateo, CA 94401

Mr. James Beck Associates
4216 Los Palos Avenue
Palo Alto, CA 94306

Mr. Walmer (Jerry) Strope
Center for Planning and Research
5600 Columbia Pike - Suite 101
Bailey's Crossroads, VA 22041

Mr. Edward L. Hill
Research Triangle Institute
P.O. Box 12194
Research Triangle Park, NC 27709

Harvey G. Ryland
Ryland Research, Inc.
5266 Hollister Avenue
Suite 324
Santa Barbara, CA 93111

Dr. John Cockayne
Senior Scientist
Science Applications, Inc.
1710 Goodridge Drive
P.O. Box 1303
McLean, VA 22101

Stan Martin & Associates
860 Vista Drive
Redwood City, CA 96062

Bell Telephone Laboratories
Attn: Mr. E. Witt
Mr. R. May
Mr. J. Foss
Whippany Road
Whippany, NJ 07981 (3)

Mr. Raymond Alger
SRI International
333 Ravenswood
Menlo Park, CA 94025

Mr. Jud Leech
BDM Corporation
1801 Randolph Road, S.E.
Albuquerque, NM 87106

Mr. Joseph E. Minor
Texas Technological College
Lubbock, TX 79408

Dr. Ben Sussholz
R1/2094
TRW
One Space Park
Redondo Beach, CA 90278

Mr. Anatole Longinow
IITRI
10 West 35th Street
Chicago, IL 60616

University of Florida
Civil Defense Technical Services Center
College of Engineering
Department of Engineering
Gainesville, FL 32601

Professor R.K. Pefley
University of Santa Clara
Santa Clara, CA 95053

RESEARCH ON UPGRADING STRUCTURES FOR HOST AND RISK AREA SHELTERS — Unclassified

September 1982

Scientific Service, Inc., Redwood City, CA
Contract EMW-C-0686, Work Unit 1128A — 227 pages

This report presents a summary of the work conducted during the first year of a five-year program. This research effort provides the engineering basis and guidance for the development of upgrading for host and risk area shelter spaces.

This investigation is in support of current Civil Defense planning based on a policy of crisis relocation, and includes investigative efforts related to glulam timber beams, concrete connections, punching strength of reinforced concrete slabs, and static/dynamic testing of prestressed concrete slabs.

The results of this study are being used in the development of a prediction methodology for comparative selection of shelter spaces.

RESEARCH ON UPGRADING STRUCTURES FOR HOST AND RISK AREA SHELTERS — Unclassified

September 1982

Scientific Service, Inc., Redwood City, CA
Contract EMW-C-0686, Work Unit 1128A — 227 pages

This report presents a summary of the work conducted during the first year of a five-year program. This research effort provides the engineering basis and guidance for the development of upgrading for host and risk area shelter spaces.

This investigation is in support of current Civil Defense planning based on a policy of crisis relocation, and includes investigative efforts related to glulam timber beams, concrete connections, punching strength of reinforced concrete slabs, and static/dynamic testing of prestressed concrete slabs.

The results of this study are being used in the development of a prediction methodology for comparative selection of shelter spaces.

RESEARCH ON UPGRADING STRUCTURES FOR HOST AND RISK AREA SHELTERS — Unclassified

September 1982

Scientific Service, Inc., Redwood City, CA
Contract EMW-C-0686, Work Unit 1128A — 227 pages

This report presents a summary of the work conducted during the first year of a five-year program. This research effort provides the engineering basis and guidance for the development of upgrading for host and risk area shelter spaces.

This investigation is in support of current Civil Defense planning based on a policy of crisis relocation, and includes investigative efforts related to glulam timber beams, concrete connections, punching strength of reinforced concrete slabs, and static/dynamic testing of prestressed concrete slabs.

The results of this study are being used in the development of a prediction methodology for comparative selection of shelter spaces.

RESEARCH ON UPGRADING STRUCTURES FOR HOST AND RISK AREA SHELTERS — Unclassified

September 1982

Scientific Service, Inc., Redwood City, CA
Contract EMW-C-0686, Work Unit 1128A — 227 pages

This report presents a summary of the work conducted during the first year of a five-year program. This research effort provides the engineering basis and guidance for the development of upgrading for host and risk area shelter spaces.

This investigation is in support of current Civil Defense planning based on a policy of crisis relocation, and includes investigative efforts related to glulam timber beams, concrete connections, punching strength of reinforced concrete slabs, and static/dynamic testing of prestressed concrete slabs.

The results of this study are being used in the development of a prediction methodology for comparative selection of shelter spaces.